OPTIMIZATION OF DRIFT CONTROLLED TYPE 2 STEEL TIER FRAMES

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ABSTRACT OF THESIS

OPTIMIZATION OF DRIFT CONTROLLED
TYPE 2 STEEL TIER FRAMES

This study develops a method for optimizing the distribution of structural steel in mid- and highrise braced framing systems to satisfy stiffness requirements as necessary to limit excessive lateral drift. This optimization procedure incorporates a term designated as the "deflection influence parameter", defined as the member deflection contribution divided by the member volume. Members with the highest deflection influence parameters are the least efficient in providing structural stiffness, and are therefore modified by the optimization routine which systematically increases the appropriate member areas. The proposed method, although developed for determinate pinned trusses, gives very close to optimal solutions for indeterminate single-bay braced frames. A computer program capable of performing this optimization routine for structures with many members is also presented, together with specific design examples selected to demonstrate the simplicity and efficiency of the method.

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CHAPTER I

INTRODUCTION

The control of lateral drift is a major design concern for modern mid- and highrise structures, particularly as building dead weights decrease and height-to-width ratios increase. In the design of many high narrow buildings, the designer finds that a braced frame with members adequately sized for strength considerations results in a structure which drifts excessively under lateral loading. When this occurs, selected member areas must be increased to achieve the required structural stiffness.

The objectives of this investigation are to develop a method for optimizing the distribution of materials to satisfy structure strength and stiffness requirements, to develop a computer program which will utilize this optimization routine for structures with many members, and to present the basic operation of and principles involved in the optimization method in a meaningful way so that the ideas encompassed can be easily understood and adopted by the designer of tomorrow's steel structures.

A review of current design practices indicates that the approaches and procedures for designing braced framing systems differ among design firms and textbooks. Most approaches and procedures include approximations and simplifications necessary to make design in the pre-computer era practical. The application of the computer's computational speed and capacity to aid in the design of braced framing systems has not been explored to the same degree as has those for rigid framing systems.

As an introduction to the optimization routine, a general review of lateral loads and building deflections is presented in Chapter II. This chapter discusses reasons for limiting excessive lateral drift in multistory buildings and provides an examination of the various deflection indices currently employed by today's designers. Chapter III provides a review of the design and analysis of braced framing systems. Common methods of providing bracing in mid- and highrise steel structures are presented, in addition to common assumptions and approximations often utilized in their design.

Following this review, Chapter IV presents the basic optimization method in a general and qualitative manner. Once the optimization procedure has been developed, two simple design examples, selected to best demonstrate the effectiveness and simplicity of the routine, are shown. Chapter V examines the topic of further research needs relevant to this area of structural engineering.

The Appendices include information on the computer program written to perform the optimization according to the method presented in Chapter IV, and provides the designer with the information necessary to computerize the optimization routine. Appendix A is a users' manual for Program WTD1, complete with a discussion of the program's capabilities and data file structure. Appendix B examines the operating sequence and flow of control for Program WTD1 and is intended primarily for the designer wishing to create a modified version to suit specialized needs. A listing of Program WTD1 is also contained in Appendix B. Appendix C presents three individual design examples which are provided to familiarize the designer with the program's current operations and procedures.

It should be noted that the majority of the information presented in this investigation concerns Type 2 steel planar midrise tier frames. For design purposes, static or "quasi-static" wind loadings have been assumed.

CHAPTER II

LATERAL LOADS AND BUILDING DEFLECTIONS

The design of multistory structures to resist lateral loads presents many interesting and complex problems. In the past, the design for lateral drift has often been secondary to the design for gravity loads. However, as building dead weights decrease and height-to-width ratios increase, the effects of lateral forces become increasingly more significant, and may, in fact, control in the design.

The purpose of this chapter is:

(1) to present a general review of lateral loads,

(2) to examine, in a general way, the structural response to these lateral loads,

(3) to define the deflection index and present typical values, and

(4) to discuss related considerations and approximations often encountered in design practice.

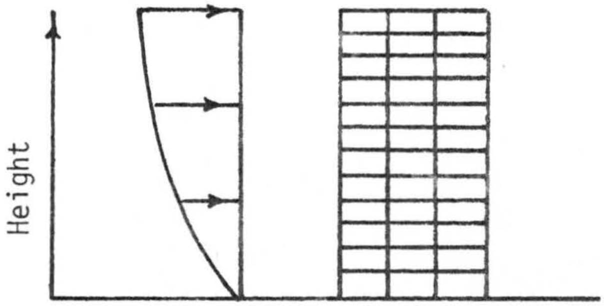
2.1 Wind Loads

The response of a multistory structure to wind forces depends upon a number of factors, including the shape of the structure, the influence of the terrain and nearby structures, the lateral stiffness provided by the structure, the velocity and density of the air, and the direction of the wind with respect to the building orientation.

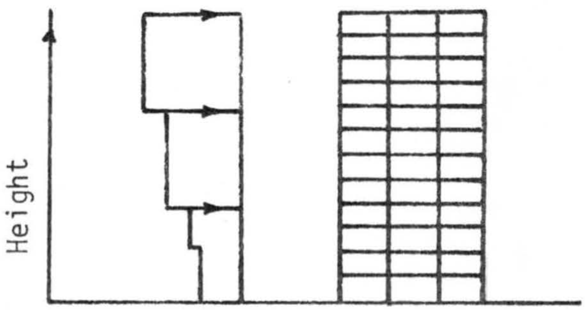
Wind design is inherently a complex dynamics problem, although most current designs are based on consideration of the wind as a static lateral load. Mean wind pressures, known to increase with height, are often idealized by continuously increasing pressure curves. For design purposes, these dynamic wind pressure curves are typically reduced to step wise statically applied pressures. It is further assumed, for simplified design load determination, that these static wind pressures may be resolved into horizontal wind forces acting at each floor level on the upwind face of the structure (see Fig. 2.1a, b, and c).

Building codes, such as the Uniform Building Code (UBC) and the American National Standard Building Code (ANSI), provide maps and charts specifying recommended wind pressures based upon the structure's geographic locality and its height above ground. Obviously, the code prescribed methods are quite general and approximate since buildings vary widely in their shapes and configurations, no two structures share the same physical environment, effects of adjacent buildings differ as additional buildings are constructed, altered, or replaced by different structures, winds vary greatly in terms of velocity, and wind directions do not remain constant, even for a specific geographic location.

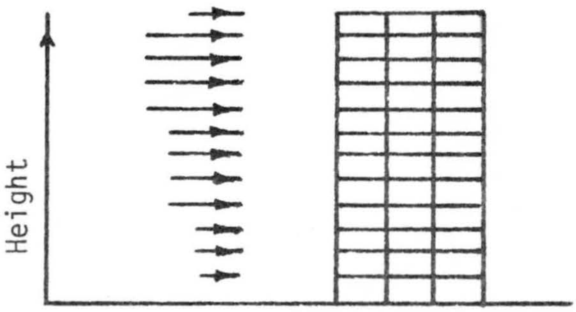
If the particular structure is of unusual height, configuration, or purpose, or if the specific geographic location experiences unusual wind speeds and/or distributions, it is advisable to perform wind tunnel studies to determine the particular static and dynamic wind characteristics.



(a)



(b)



(c)

Figure 2.1 Mean dynamic wind pressures (a), assumed static wind pressure distribution (b), and static wind forces (c).

2.2 Earthquake Loads

Potentially destructive earthquakes can occur in nearly every region throughout the United States. It is for this reason that seismic forces should be considered in most structural designs.

Earthquake loads are caused by structural distortions induced in the structure by the surrounding ground motions (see Fig. 2.2). The displacements, velocities, and accelerations produced by an earthquake are erratic and generally unpredictable in direction, magnitude, time of occurrence, and sequence.

At present, two basic approaches exist for estimating earthquake loads: the quasi-static method and the dynamic analysis. The quasi-static approach, developed by the Structural Engineers Association of California (SEAOC) and incorporated in the UBC and ANSI codes, simulates earthquake loading through the calculation of static horizontal forces. The basic SEAOC formula, which intends to produce a structure that will sustain light earthquakes with no structural damage or inelastic action, incorporates a number of coefficients and factors to account for the seismic risk or zone factor, the type or arrangement of the structural resisting elements or horizontal force factor, the building's fundamental period of vibration, and the total building dead load. The 1974 revised SEAOC formula, adopted in the 1976 UBC, incorporates, in addition to those previously mentioned, an occupancy importance factor and a soil amplification factor, and adds a fourth and more severe seismic risk factor for parts of the California-Nevada region.

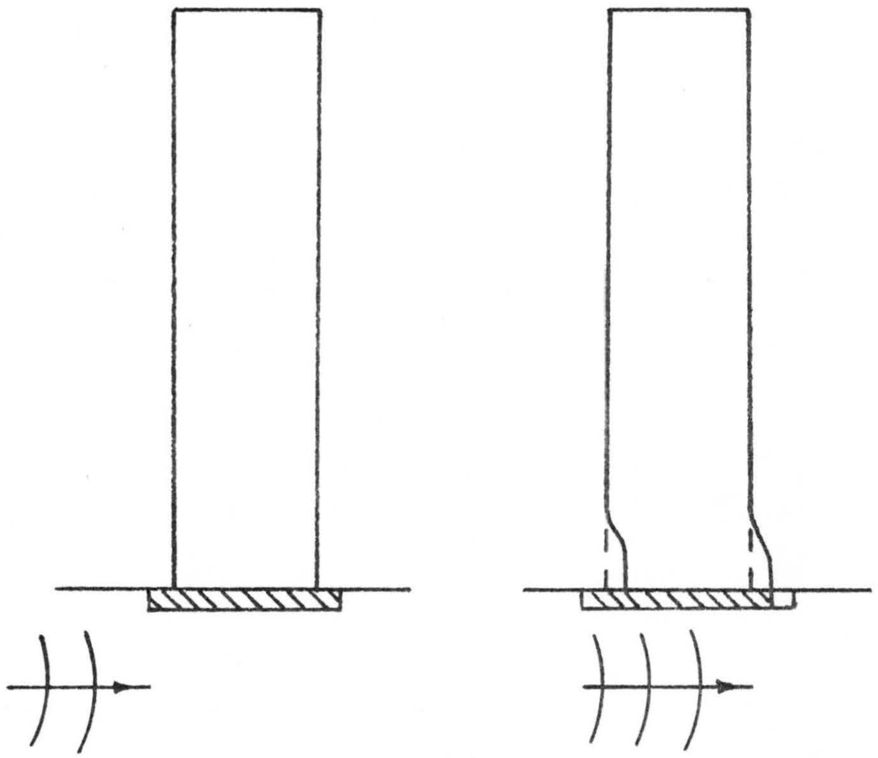


Figure 2.2

Earthquake ground motion.

The quasi-static approach, although quite popular because of its relative simplicity, is extremely approximate as it is virtually impossible to rationalize this type of dynamic problem into an equivalent static problem.

The dynamic analysis, becoming increasingly more popular due to recent computer techniques and applications, and which is required for major structures by some codes, idealizes the structure as an assembly of masses interconnected by springs and dampers. The dynamic analysis can be based on an elastic response, or can be performed to account for the structure's inelastic response. The inelastic dynamic analysis more precisely predicts the forces generated in the remainder of the structure, but such methods are too complex for most designs. Problems with the dynamic method of analysis are that the analysis programs are quite sophisticated, requiring much background knowledge and understanding, and the earthquake and structural response characteristics are generally difficult to estimate.

2.3 Structural Response to Lateral Loads

Many times, multistory structures adequately designed for strength considerations drift excessively when subjected to lateral loads. Control of this lateral drift is therefore imperative to limit perceptible building motion or sideway, to prevent damage to nonstructural elements such as plaster walls and glass, and to avoid the damage or failure of equipment such as elevators and certain business machines.

When subjected to lateral loads, a structure has the tendency to oscillate or vibrate. Typically, under wind loading a building will vibrate (or respond) primarily in the first mode, while earthquake loads usually cause more significant contributions from the higher modes of vibration. Throughout the duration of the lateral loading, the structural deflection at the topmost level will vary in a seemingly random manner; possibly, during severe loadings, exceeding the specified design drift limit (see Fig. 2.3).

2.4 Deflection Indices

Structural engineers typically use the term "deflection index" to indicate a particular structure's drift characteristics. Two interpretations of the deflection index exist; some designers interpret it as the individual story deflection divided by the story height (also known as the inner story drift), while others define it as the total structure drift divided by the total structure height (see Fig. 2.4a and b).

For design purposes, the choice of a reasonable deflection index is extremely important. If too high an index value is selected, the building will sway excessively under lateral loads, while, if too low an index value is selected, system inefficiency in the form of unnecessarily high steel costs will result.

Gaylord and Gaylord (1968) present a number of factors which should be considered in the selection of a reasonable deflection index:

(1) the type of building use and occupancy, as enough lateral stiffness must be provided to prevent perceptible building motion, plaster cracks, and sway noises,

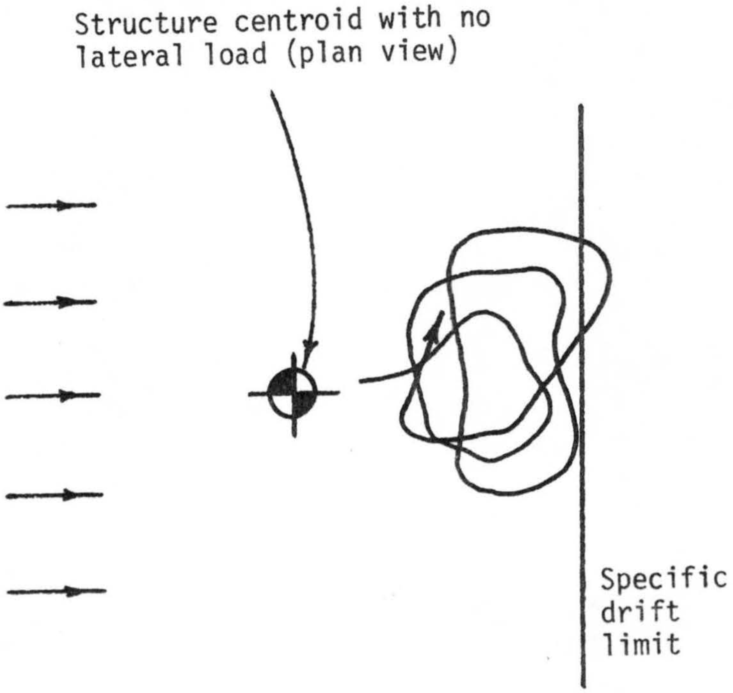


Figure 2.3
Motion of building centroid
caused by wind forces.

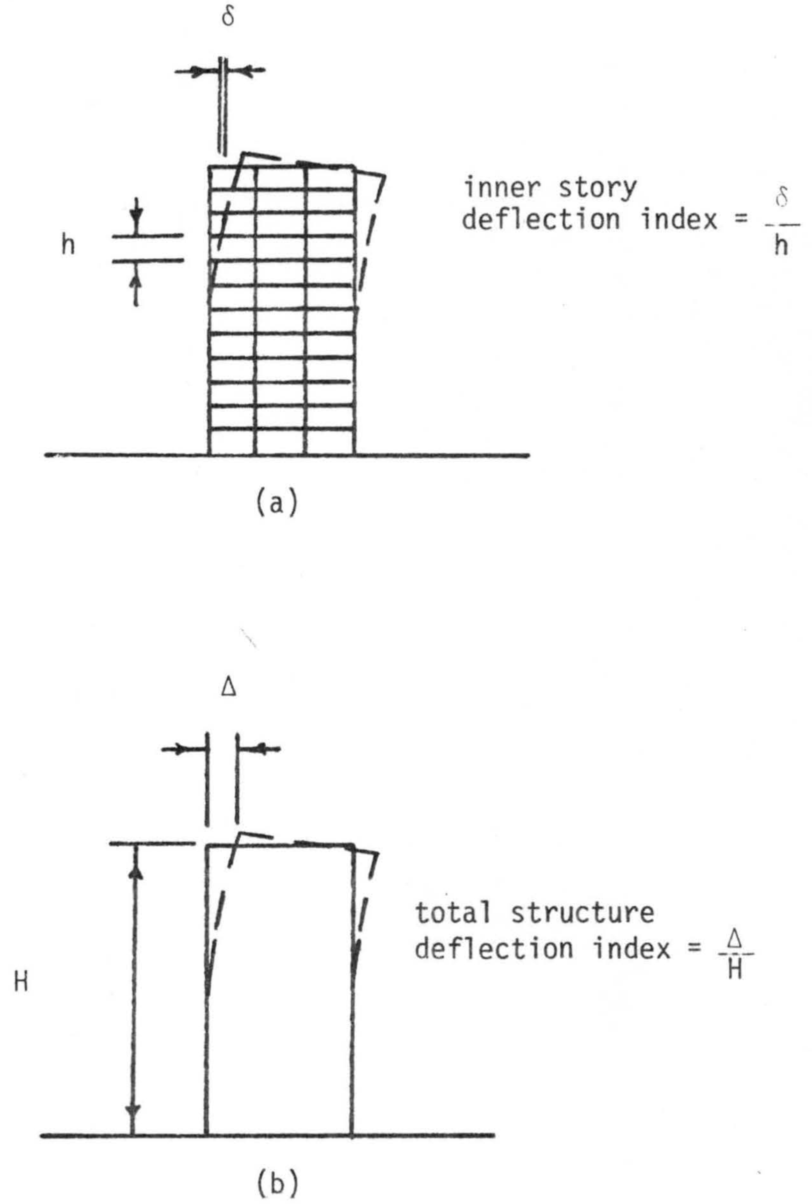


Figure 2.4 Inner story deflection index (a) and total structure deflection index (b).

(2) the stiffening effects of interior and exterior walls and floors, which actually provide additional lateral stiffness to the structure and are often otherwise ignored in the deflection computations,

(3) the possible exterior shielding against wind loads, as may be provided by surrounding buildings, and

(4) the magnitudes of the design wind forces, as some building codes may require higher standards with respect to strength and stability criteria than may be deemed reasonable by the designer.

2.5 Typical Deflection Indices

To demonstrate the subjectivity in establishing a deflection index, the following sources have been reviewed:

(1) The Uniform Building Code (1973) does not specify deflection indices, stating that lateral deflections should be considered in accordance with accepted engineering practice.

(2) The National Building Code of Canada (1970) arbitrarily limits both the inner story deflection and the total building deflection to 1/500 in buildings with height-to-width ratios greater than four. The NBC further states that these limits may be waived if the design is based on a detailed dynamic analysis of the deflections and their effects.

(3) The Structural Engineering Handbook (Gaylord and Gaylord, 1968) lists inner story drift indices ranging from 0.0015 to 0.003, depending on the type of building, the type of construction, exposure, and code wind requirements.

(4) The second edition of Structural Steel Design (Tall, 1974) references an American Society of Civil Engineers (ASCE) Committee study which recommends limiting overall structural deflections to 0.002 times the height of the building. The Committee study further noted, however, that buildings with deflection indices exceeding 0.005 have performed satisfactorily.

(5) The Structural Steel Designer's Handbook (Merritt, 1972) references a SEAOC report which suggests limiting the inner story drift to 0.0025 for wind, and 0.005 for earthquake. The smaller allowable drift for wind accounts for several factors: wind motions occur many times in the life of the structure and should be limited for occupant comfort, during a severe earthquake the safety of the occupants and the survival of the building and its contents, rather than personal comfort, are of primary concern. Inelastic response is considered permissible for earthquake loading and will reduce the response computed using an elastic analysis; however, inelastic behavior should not result from wind loads except those caused by tornadoes and very extreme hurricanes.

(6) Foreman, in her thesis "Wind Drift Criteria Currently Employed in Tall Building Design" (1975), presents a review of drift indices obtained both from current literature and engineering firms which design multistory structures. Deflection indices ranged from 0.001 to 0.005 depending upon the particular structural application. The most commonly

utilized index values fell in a range from 0.0014 to 0.004. Inner story drift indices were not so widely spread and commonly varied from 0.0015 to 0.003.

Obviously, the computed drift is extremely dependent upon the assumptions and approximations used in the lateral load deflection analysis. These assumptions and approximations have not and probably never will be standardized. Higher deflection indices can be tolerated as the deflection calculations become more conservative in the assumptions employed. For example, larger computed deflections would be allowable if the contributions of nonstructural elements and the inelastic response of an earthquake loaded building were neglected rather than if the actual building behavior was fully modeled.

CHAPTER III

THE DESIGN AND ANALYSIS OF BRACED FRAME SYSTEMS

Many methods of providing lateral strength and stiffness exist for structures subjected to lateral forces. Certain design assumptions and approximations are typically used with each method. Each method also has both advantages and disadvantages limiting its practical application. Most importantly, the designer must know and understand the behavior of the selected system to insure a functional structure.

The purpose of this chapter is:

1) to define, in basic terms, a braced framing system,
2) to present and explain a typical braced frame design,
3) to examine several of the more common forms of lateral bracing used in low-, mid-, and highrise structures, and
4) to discuss the braced frame deflected shape and present structure drift computations.

3.1 Braced Framing Systems

The American Institute of Steel Construction's "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," (AISC, 1969) defines three basic types of construction, each dependent on the design and behavior of the beam-to-column connections. These basic types of construction and their respective design assumptions are:

1) Type 1, or "rigid frame" construction, which assumes that the beam-to-column connections remain rigid or unchanged with respect to the original beam-to-column angles,
2) Type 2, or "simple frame" construction, which assumes that the beam-to-column connections are pinned or free to rotate under gravity loading, and
3) Type 3, or "semi-rigid" framing, which assumes that the beam-to-column connections possess a dependable and known moment capacity.

Both the Type 1 and Type 2 methods of construction represent idealized models of what actually occurs in a structure. Rarely, if ever, would any beam-to-column connection remain fully fixed with no angle rotation, or conversely, totally simple with no moment transfer. These two methods of construction, while based upon certain important connection behavioral assumptions, permit the more rapid and efficient design and analysis of steel framed structures.

Braced frames are generally designed with Type 2 or simple beam-to-column connections. For design purposes, each connection or joint is assumed to be a frictionless pin, resulting in what is often termed a "pin-connected structure." In actuality, these connections are typically bolted and have some moment capacity; thus their behavior approaches that of the Type 3 connections. This is recognized in the AISC Code, Section 1.2, by provisions which allow Type 2 connections to be assigned wind moments when adequate connection strength and rotation capacity exist. Because these connection characteristics are difficult for the designer to determine, Type 2 connections are not often assigned moments from lateral loads.

Theoretically, an ideal unbraced Type 2 pinned portal frame is laterally unstable. Any lateral force of any magnitude will cause structural instability. For this reason, some form of lateral bracing must be incorporated into the frame to provide the required lateral strength and structural stability (see Fig. 3.1a and b).

Thus, a braced frame acts to resist all assigned gravity and lateral loads in addition to providing structural stability and the structural stiffness required to limit excessive lateral deflections.

3.2 Structural Design Procedure

The structural system in a typical multistory steel framed building consists of beams, girders, and columns, all of which must be designed to control working load stresses and to provide adequate stability and sufficient stiffness for drift control. In formulating a structural design procedure, it is imperative for the designer to visualize the manner in which the structure will resist the applied loads. As all structures must be designed to support the anticipated gravity loads (dead and live), this would seem to be the natural first step in the design procedure. After the basic building system, including the lateral load resisting system, is chosen, lateral stability is initially assumed and the lateral support system is added and designed later in the procedure. The lateral load resisting system is given more and earlier attention as the building becomes taller or more slender and as the lateral forces assumed in the design increase. The fundamental braced frame design approach for most low-, mid-, and some highrise structures consists of:

1) selecting the structural system, column spacings, and the lateral load resisting system,

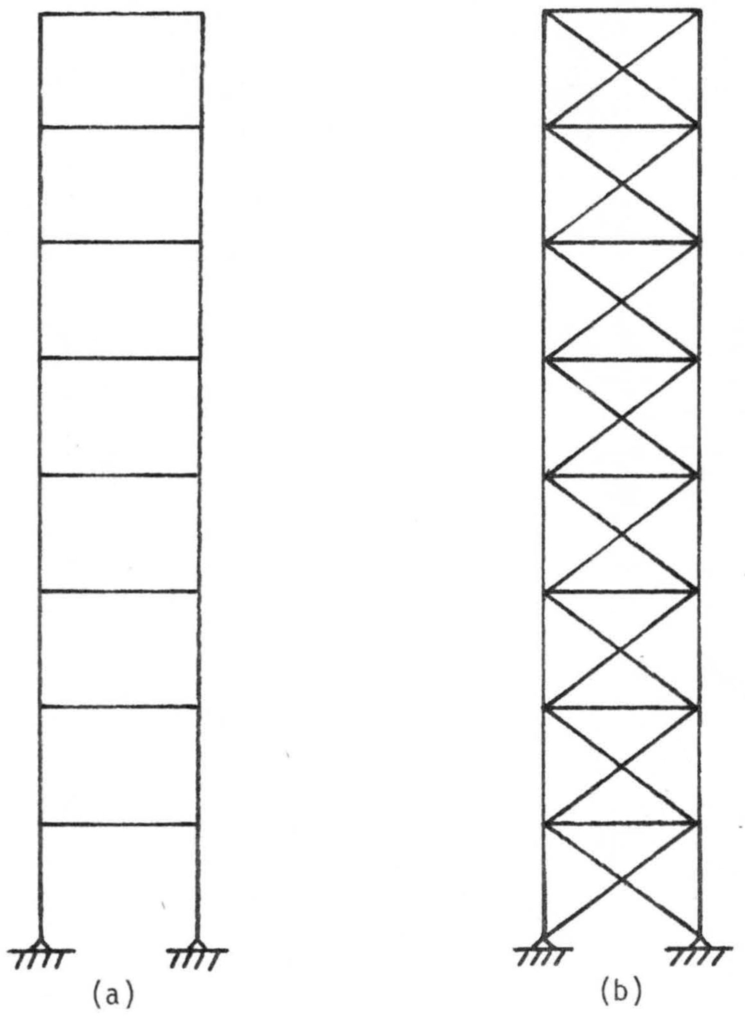


Fig. 3.1 Type 2 frames, laterally unstable (a), and a laterally stable X-braced frame (b).

2) designing the floor system for gravity loads,
3) establishing preliminary beam and girder sizes based on the gravity loads,
4) establishing the preliminary column sizes based on the gravity loads,
5) conducting a lateral load analysis,
6) modifying member areas if overstressed,
7) checking the structure for excessive lateral drift, and,
8) modifying selected member areas as necessary so that the maximum permissible structure deflection is not exceeded.

For combined loading cases, such as gravity loads and wind or earthquake forces acting simultaneously, AISC Section 1.5.6 permits an allowable stress increase of 33 percent. The final member selection should be based on the more critical of either the gravity load design or the combined load design. The effect of the 33 percent stress increase allowed for lateral loads is often obtained by using the gravity load allowable stresses and 75 percent of the combined loads. When this procedure is followed, the designer must insure that the full design lateral loads are used in the deflection computations.

Many mid and highrise structures previously designed for strength adequacy drift excessively when subjected to the design wind forces (see Design Example 2). Additional steel is therefore required to increase the structure's lateral stiffness. The quantity of additional structural steel required for a particular building is a measure of the system efficiency. The wind premium concept, often utilized in comparing system efficiences, is based on the additional steel required, above that required to resist gravity loads, to resist lateral loads and limit excessive lateral drift (see Fig. 3.2). In Figure 3.2, the rising gravity steel curve results from the additional column steel required to support the increasing number or floors. Note that the wind premium increases nonlinearly with increasing structure height. In general terms, the lower the wind premium with respect to the gravity steel required, the more efficient is the structure.

With the many structural analysis programs available today, it is a fairly routine procedure to check member strength adequacy for the design load cases. However, with increasing emphasis on structural efficiency and optimization, computer oriented design routines are becoming increasingly more widespread and popular. With the computer's high speed and storage capacity, it may soon be possible to design and optimize large structures for any number of design criteria, including gravity loading cases, wind loading cases, combined loading cases, and even structure drift limitations. Most design programs attempt to optimize a structure based on least steel weight criteria, performing design-analysis iterations until such time that a least weight structure satisfying all design constraints, is obtained.

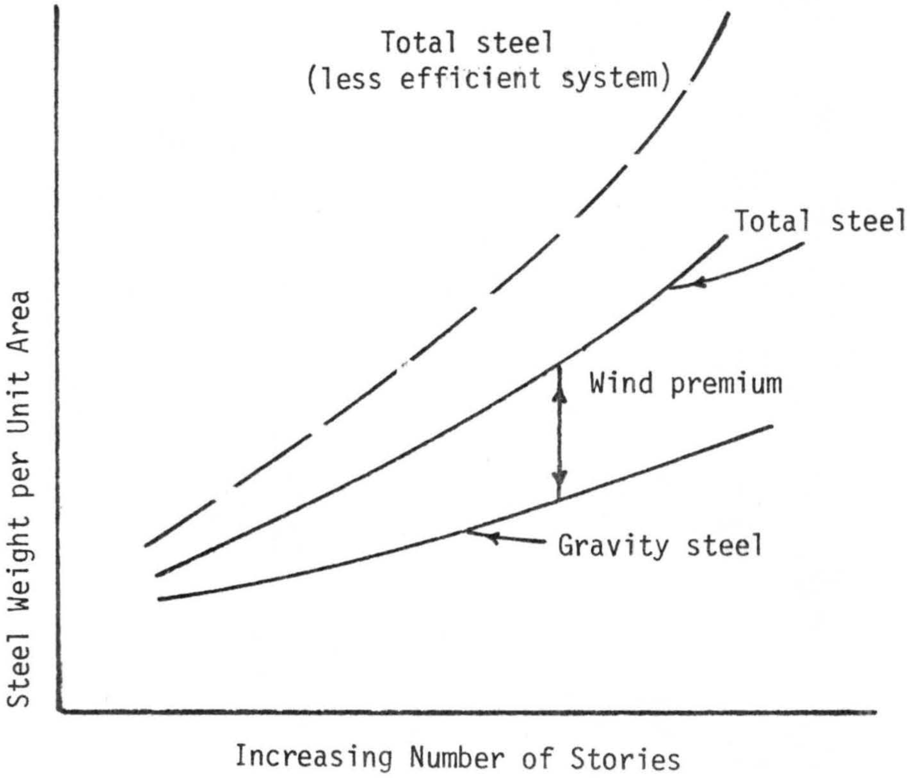


Fig. 3.2

Wind premium concept.

3.3 Types of Lateral Bracing

Single bay braced framing arrangements utilized to resist lateral forces in multistory steel structures include X-bracing, K-bracing, full story knee-bracing, and many alternate forms of bracing.

The X-bracing configuration is an efficient and popular method of providing lateral load resistance (see Fig. 3.3). Two variations of X-bracing designs exist, the statically determinate "counter" system and the statically indeterminate "fully effective truss." With the counter system, only the tension diagonal is assumed to act in resisting lateral forces. The compression diagonal, with a large l/r ratio, is assumed to buckle, contributing no resistance. The fully effective truss has members designed to sustain either compression or tension loads. The counter X-bracing system is typically employed in lowrise and warehouse type structures (see Design Example 1, Appendix C), while the fully effective truss is most commonly utilized in mid- and highrise structures where all of the load supporting and resisting elements are generally quite large (see Design Example 2, Appendix C).

X-bracing design and analysis assumptions and considerations include the following:

1) Both variations of X-bracing obstruct an entire bay within the structure. For this reason, it is usually most convenient to place the bracing around elevator shafts, stairway walls, and other core areas.
2) For a fully effective truss, both bracing diagonals must be capable of resisting compression loads without buckling.

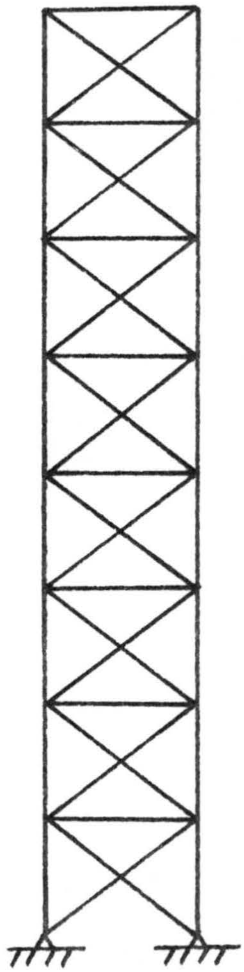


Fig. 3.3
X-braced frame.

3) If the bracing diagonals are connected at their intersection, their unsupported length for in-plane buckling is halved.

4) Column shortening due to gravity loads and structure drift cause additional stresses in the bracing members. This effect is typically ignored in first order analyses.

5) In mid-and highrise structures, gravity and wind loads acting in combination often control in the design of non-bracing members. Especially affected are the lower story columns, which should be checked for these increased loads, and the lower story beams, which should be examined for beam-column interaction resulting from increased axial forces.

K-bracing, or inverted V-bracing, is another efficient and commonly used method for providing lateral load resistance (see Fig. 3.4). With this configuration both bracing members must be designed to act in compression, since this bracing supports a major portion of the floor beam gravity load, as well as resisting the applied lateral loads. K-bracing is typically employed in mid-and highrise structures where all of the load supporting and resisting members are generally quite large (see Design Example 3, Appendix C).

K-bracing design and analysis assumptions and considerations include the following:

1) Although being more adaptable to achitectural restrictions than is x-bracing, the K-braced truss still obstructs the use of much of the entire bay within the structure.

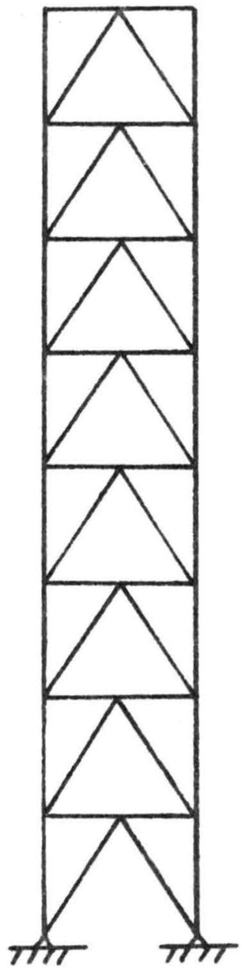


Fig. 3.4
K-braced frame.

2) Bracing members must be designed to accept the overlying beam loads in addition to the applied lateral loads. The floor beam is typically designed as a two-span continuous beam.

3) Column shortening due to gravity loads and structure drift cause additional stresses in the bracing members. A first order analysis which ignores these secondary stresses is very often used.

4) The columns and beams in the lower stories of mid-and highrise K-braced frames should be checked for the increased stresses due to gravity and wind loads acting together.

The full story knee-bracing configuration differs from the K-braced frame in that the bracing members are separated and thus do not share a common floor beam joint (see Fig. 3.5). This feature serves to create more useable space for architectural and mechanical purposes. The floor beams in the braced frame must have adequate flexural strength for this system to function. The K-bracing design and analysis assumptions and considerations also apply for the full story knee-bracing, however, increased bending stresses will occur in the lower floor beams as a result of increased wind forces in the bracing members.

Alternate single bay bracing configurations are occasionally utilized or developed to meet specific structural, architectural, or mechanical requirements. Some of these systems include diamond bracing, useful where window openings are desired, K-bracing with a common column joint instead of a common beam joint, and V-bracing, where the

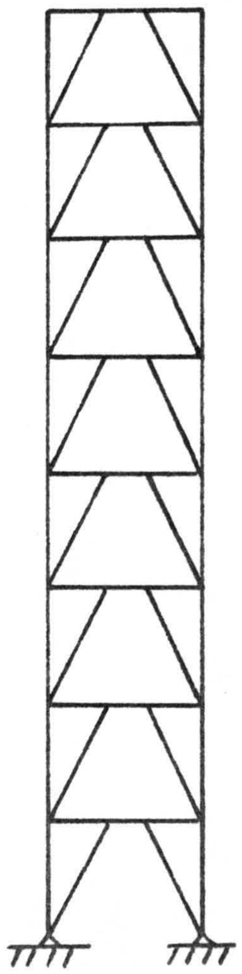


Fig. 3.5

Full story knee-braced frame.

bracing members support the floor below (see Fig. 3.6a, b, and c). Each of these systems has a different set of design and analysis assumptions, in addition to specific advantages and disadvantages, all of which must be considered in the selection of a braced frame configuration.

3.4 Braced Frame Deflection Shape and Drift Computation

The deflected shape for a single bay braced framing system is shown in Fig. 3.7. Braced frames demonstrate this flexural mode of deformation provided that the structure has a high height-to-width ratio, lateral loads are uniform on all levels of the structure, and member sizes remain constant throughout the height of the structure.

The flexural deformation of an actual structure will differ slightly from that shown as bracing members will elongate or shorten somewhat as a result of structural shear deformation, lateral design loads seldom remain uniform with increasing height, and finally, member areas will typically increase from structure top to bottom as a result of increased gravity and wind loads.

The lateral drift of a braced frame is computed as the sum of all individual member deflection contributions, or as more commonly written from the flexibility or unit load method of structure analysis:

$$\Delta \text{ total} = \Sigma \frac{NnL}{AE} \tag{3.1}$$

where Δ = the total structure deflection at the point and in the direction of the applied unit load,

N = the tensile or compressive force in the individual member due to the real loading system (tensile

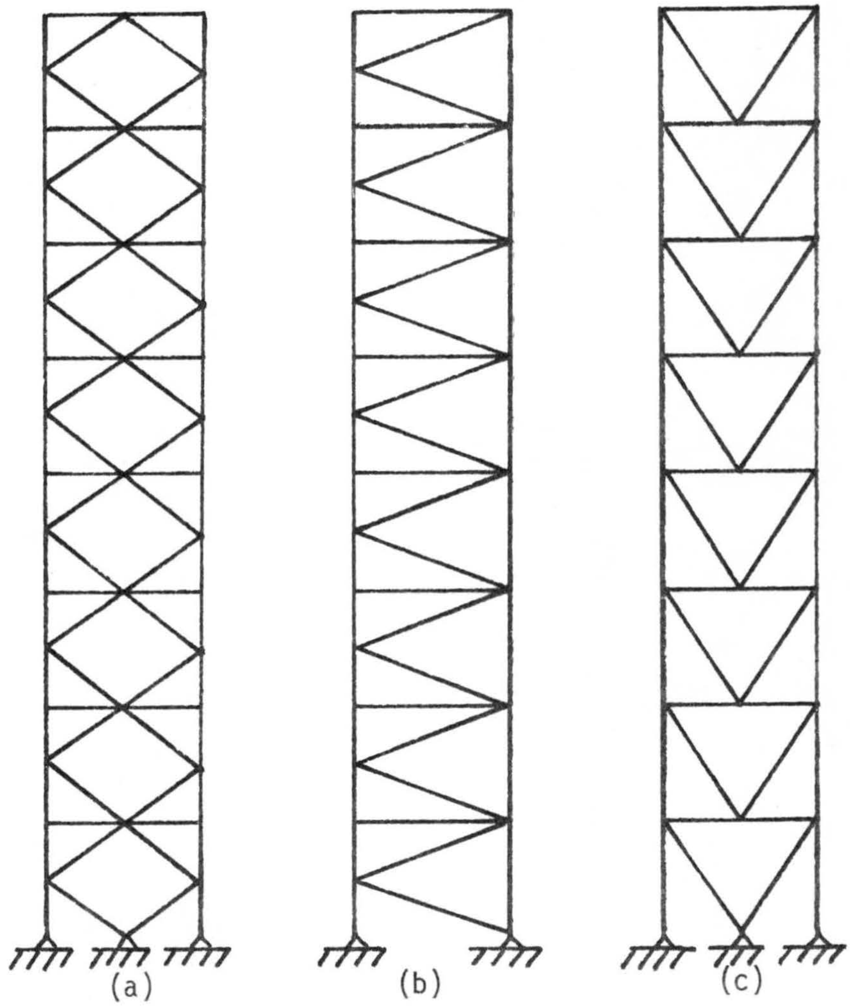


Fig. 3.6 Alternate bracing configurations, diamond bracing (a), K-bracing (b), and V-bracing (c).

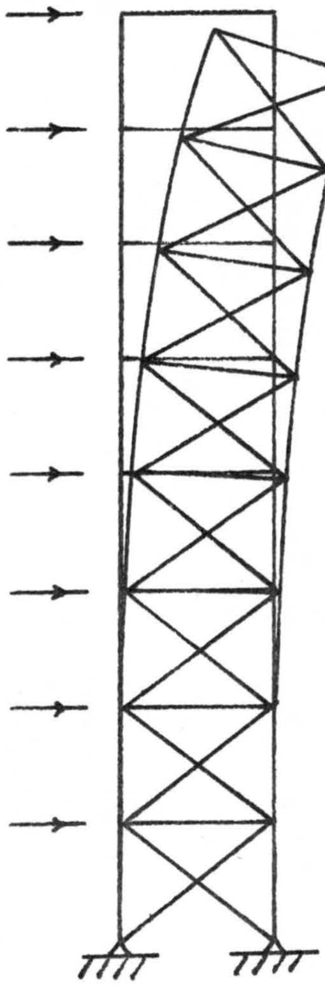


Fig. 3.7
Braced frame deflected shape.

forces are typically assumed positive while compressive forces are assumed to be negative),

n = the tensile or compressive force in the individual member due to the unit loading system (dimensionless),

L = the individual member length,

A = the individual member cross-sectional area,

E = the individual member modulus of elasticity.

The real and unit load member forces may be determined from simple statics if the structure is statically determinate. Otherwise an indeterminate structural analysis is required to generate the appropriate member forces.

When analyzing a braced frame for excessive lateral drift, it is generally most convenient to create a table listing the member number, member type (column, beam, or bracing member), the real and unit load assigned to the member, and the member properties, length, area, and modulus of elasticity. The individual member deflection contributions may then be calculated and recorded in another column and summed to give the total lateral drift at the point of the unit load. Lateral deflection due to deformation of the foundation and soil below the building is usually ignored, but can be modeled by springs or members assigned appropriate member stiffness factors.

Should the maximum allowable lateral drift be exceeded, as occurs in many mid- and highrise structures adequately designed for strength considerations, the designer must increase the areas of some members to reduce the member deflection contributions and thereby reduce the total structure deflection. At present, the members to be increased and the pattern of increases to be used are determined by the designer based on his experience, an examination of the deflection contributions from each member or group of members, and by repeated trials.

In the following chapter, a systematic and optimal approach and solution to the dilemma of member area modification and lateral deflection reduction will be presented and discussed.

CHAPTER IV

THE DEFLECTION INFLUENCE PARAMETER

As noted in the previous chapters, braced frames adequately designed for strength considerations may exhibit excessive sway when subjected to the full lateral design loads. If and when this occurs, the designer must increase selected member areas to reduce the individual member deflection contributions and thereby reduce the structure's computed lateral drift. The method by which these members can best be selected and the procedure by which they are modified is the prime concern of this chapter.

Although literature consisting of numerous structural steel design handbooks, special reports, committee and conference proceedings, and structural engineering articles and journals was reviewed, no information concerning the topic of optimization for Type 2 drift controlled structures was located. Most of the reports and articles dealt with rigid framing systems and structural optimization routines for stress or strength design considerations and constraints.

The purpose of this chapter is:

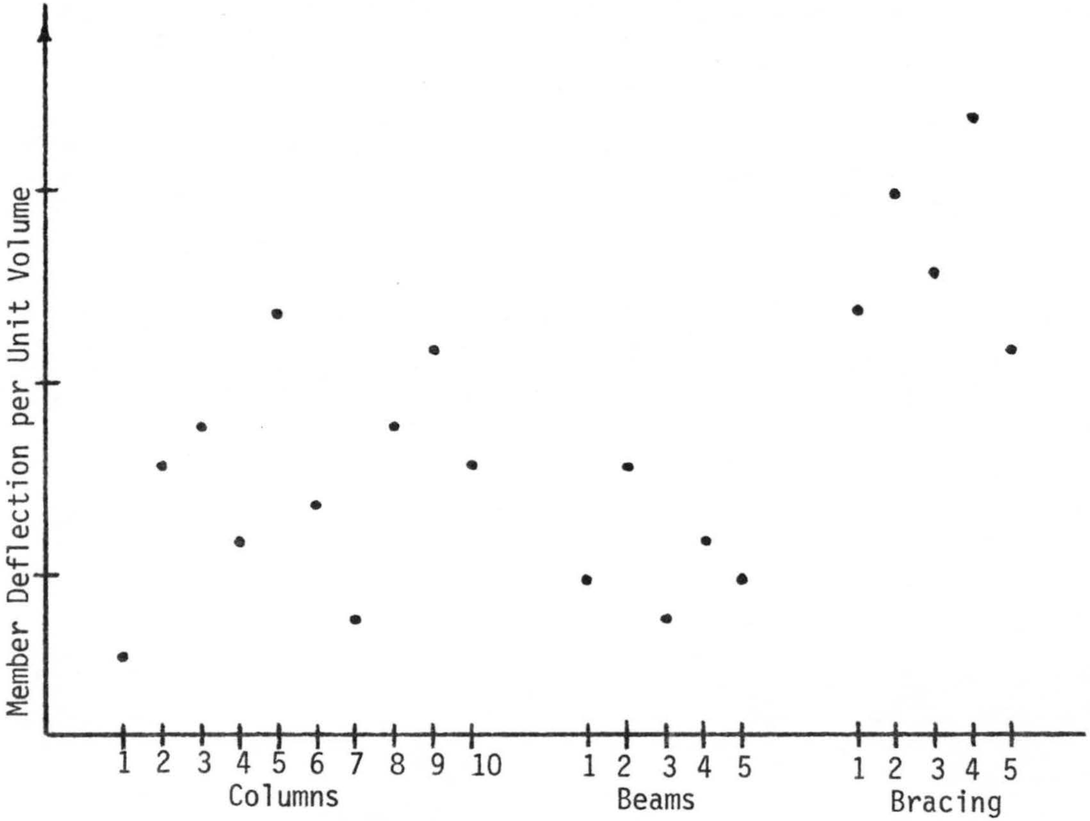
1) to develop a method for optimizing member areas as necessary to limit the computed lateral drift to an acceptable value,
2) to present the method, together with a simple design example, and

3) to introduce Program WTD1, a computerized optimization routine specifically written to execute this procedure for structures with many members.

4.1 The Deflection Influence Parameter and Basics of the Optimization Method

The relative values of the member deflection contributions, or NnL/AE terms (see Eq. 3.1), are often used as an approximate method to determine where to add more steel should the maximum allowable drift be exceeded. The members with the highest deflection contributions are typically the bracing members, having longer lengths and smaller cross-sectional areas than most other members, and the lower columns, which generally must resist the largest forces from the real loading system. While this procedure of examining the member deflection contributions to determine which areas to modify to achieve drift reduction does offer some guidance on where best to place additional steel, it is certainly not optimal (in terms of least structural steel weight) for large structures with many members and for structures where the computed drift is much greater than the maximum allowable drift.

However, if the individual member deflection contributions are divided by their respective member volumes, a deflection per unit volume term results. Obviously, the highest deflection contribution per unit volume terms represent the members least efficient in providing lateral stiffness, while the lower terms represent the members providing the greatest lateral stiffness. For any particular pinned frame, these deflection per unit volume terms may be computed and compared. The use of a plot such as shown in Fig. 4.1 is often convenient to visualize these values. An increase in lateral stiffness,



Members Active in Bracing System

Fig. 4.1

Deflection per unit volume terms.

or conversely, a decrease in the structure's lateral drift, can be achieved with the least increase in material volume if the member or members having the highest deflection contribution per unit volume are increased in area. These deflection per unit volume terms, which form the backbone of the proposed optimization procedure, are henceforth referred to as the "deflection influence parameters" and are abbreviated as the "DEFP" values.

To provide the needed structural stiffness with the least additional volume of material, the higher member deflection influence parameters must be reduced to some yet unknown limiting value. Stated in another fashion, the proposed procedure involves locating the highest deflection per unit volume level which will provide the structure with the required lateral stiffness necessary to exactly equal the maximum allowable deflection.

The successive reduction of the DEFP values can be accomplished in at least two ways. From the plot of the DEFP values and the amount of additional stiffness needed, the designer may specify, by trial and error, an estimated maximum deflection influence parameter. Members with DEFP values exceeding the trial DEFP limit are systematically increased in area to achieve the target value. The structure deflection is then recomputed and compared with the desired deflection to determine if the optimal DEFP value is higher or lower than the trial value. This procedure, although simple in concept and useful to achieve approximate drift control, has two major drawbacks; first, this procedure can involve many trials before the optimum solution is found, and second, this procedure involves judgemental decisions difficult to program for computer operations.

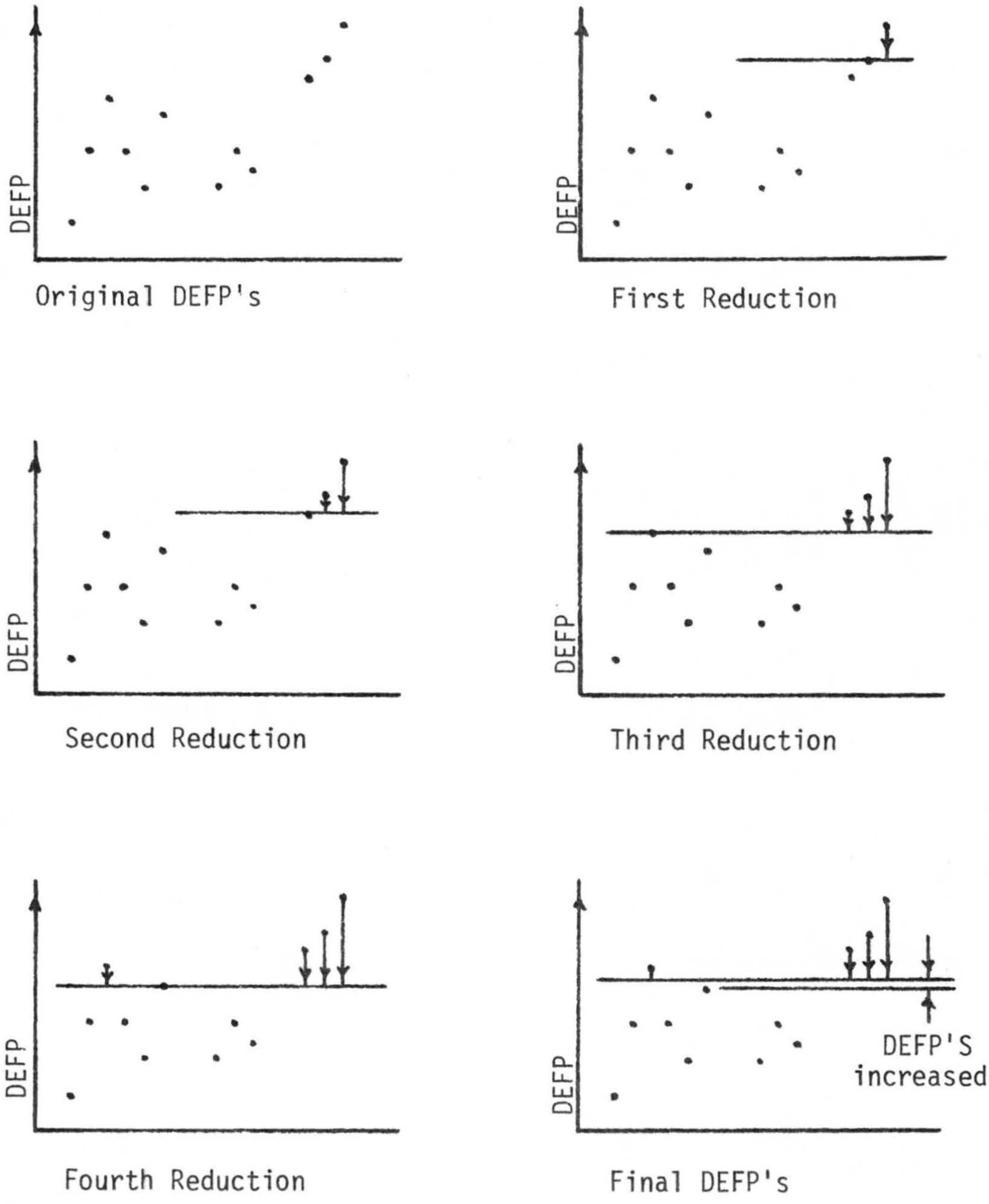


Fig. 4.2 Optimization procedure using the deflection influence parameters.
Note: Following the fourth reduction, the desired deflection exceeds that computed; the final DEFP is some, as yet, undetermined value above this limit.

The second method of successive reduction, more adaptable to computer logic, involves ordering the DEFP values in a highest to lowest fashion and systematically selecting a lower value until the computed lateral drift becomes less than or equal to that desired. This method, illustrated in Fig. 4.2, is also applicable to hand computations and is especially useful in determining the lowest required DEFP for smaller structures with few members.

4.2 Steps in the Basic Optimization Method

The first step in the proposed optimization procedure involves computing the member deflection influence parameters, and the total drift of the structure. The deflection influence parameter, defined as the member deflection contribution divided by the member volume is computed as follows:

$$\text{Deflection Influence Parameter} = \frac{NnL}{AE} \;/\; (AL) = \frac{Nn}{A^2E} \qquad (4.1)$$

Should the computed structure drift be less than or equal to the maximum allowable deflection, the design is complete and the optimiztion procedure is not required. However, should the maximum allowable drift be exceeded, some member areas must be increased. The proposed optimization routine may be easily and efficiently implemented to select those members which will provide the greatest lateral stiffness increase for the least additional structural steel.

The second step involves successively sweeping the deflection influence parameters down to a level at which the computed lateral deflection is equal to or less than the maximum allowable deflection. As previously described, this can be accomplished by systematically reducing the deflection influence parameters of all members being considered to the next lower DEFP value. In each step of the sweeping procedure, the new member area is computed, together with the new member deflection contribution and deflection influence parameter. The new member area is computed from the definition of the deflection influence parameter and is:

$$\text{Area (new)} = \text{Area (old)} \sqrt{\frac{\text{DEFP (original)}}{\text{DEFP (revised)}}} \qquad (4.2)$$

The new structure deflection may be computed as the original structure deflection minus the difference in the old and new member NnL/AE deflection contributions. The new member deflection influence parameter must be identically equal to the selected or revised DEFP; otherwise an error has been made. This procedure of member area modification is then repeated until the computed structure drift becomes less than or equal to the desired lateral deflection. Depending on the spacing of the DEFP values near the last one considered and the deflection obtained at the next to the last DEFP, the computed structure drift at the end of step two can be very close or considerably under the desired value.

The third step of the optimization method involves increasing the critical deflection influence parameter until the computed lateral drift exactly equals the maximum allowable deflection. Only members originally possessing a DEFP value greater than the last value considered in step two are modified in this procedure. This procedure is accomplished as follows:

1) compute the difference between the original and desired structure deflection,

$$\text{deflection reduction} = \Delta_r = \Delta_o - \Delta_d \qquad (4.3)$$

2) compute the summation of the original deflection contributions of all members involved in the area modification procedure,

$$\Delta_m = \text{original } \Sigma \text{ NnL/AE of modified members} \qquad (4.4)$$

3) compute the required structure deflection contribution from the modified members,

$$\Delta_n = \Delta_m - \Delta_r \qquad (4.5)$$

4) compute the summation of the most current deflection contributions of all members involved in the area modification procedure,

$$\Delta_c = \text{current } \Sigma \text{ NnL/AE of modified members} \qquad (4.6)$$

5) compute the area decrease factor required,

$$\text{ADF} = \Delta_c \;/\; \Delta_n \text{ , and} \qquad (4.7)$$

6) multiply each member area computed in the last member modification by the area decrease factor.

As a computational check, the deflection of the structure and the member deflection influence parameters should be computed using the final member areas. These calculations should verify that the desired deflection has been exactly achieved and that all modified members have the same deflection influence parameter, a value which is exceeded by no other member in the system.

Two important comments concerning the use of the deflection influence parameters follow. First, the member DEFP values must never be set above their original value, the value resulting from a stress or strength design. To do so would obviously decrease the member area below that necessary for strength adequacy. In the usual truss, many members, particularly beams and mid- and upperstory columns, will have cross-sectional areas determined by gravity load considerations. These members can display very low DEFP values and usually will not be modified by the optimization routine.

Secondly, members having negative DEFP values, if present in the system, should not be modified. For these members, a deflection reduction would require a more negative DEFP contribution. This would involve decreasing the original member area and thus lead to strength inadequacy.

4.3 Extensions of the Basic Optimization Method

The method as described in Sec. 4.2 is for determinate pinned frames in which the areas of each of the members can be varied independently. These restrictions are not always met; lateral load resisting trusses are often indeterminate and both symmetry and the usual two or three story high column lengths result in many members having to be varied as a group.

The basic method presented in Sec. 4.2 also assumes that the forces in the members remain constant, i.e. are independent of the member changes, for all steps. This condition is met if the structure is actually determinate, but obviously is not if the pinned truss is indeterminate. For the indeterminate truss, the structure must be reanalyzed after step three of optimization method is completed. Member forces will be redistributed as a result of the area modifications, and the computed lateral deflection and individual member deflection contributions and DEFP values will be changed, usually slightly, from those computed at the end of step three. If the deflection of the reanalyzed structure is inadequate, the optimization procedure can be repeated as necessary using the member forces from the latest analysis.

If two or more members are to retain an equal area following the optimization procedure, the number of possible variables is reduced. This would occur when opposing columns and bracing members must be equal in order to resist wind from either direction or when columns at adjacent levels are required to have equal areas. The individual deflection influence parameters of such groups of identical member areas must be combined together to give a group DEFP equal to the sum of the member deflection contributions divided by the volume in the group:

$$\text{Group DEFP} = \frac{\Delta_1 + \Delta_2 + \Delta_3 + \ldots + \Delta_n}{V_1 + V_2 + V_3 + \ldots + V_n} \tag{4.8}$$

where n is the number of members in the group.

4.4 Example Problem Illustrating the Method

The one-bay three-story truss system shown in Fig. 4.3 will be used to demonstrate the usefulness, effectiveness, and simplicity of the proposed optimization routine. It should be noted that this example problem was selected with the intent of illustrating the optimization technique, procedure, and concepts, and is not intended to fully portray an actual design.

The structural geometry and structure design loads are designated in Fig. 4.3. For design purposes, it is assumed that the X-bracing acts as a counter system, therefore rendering the structure statically determinate. The modeled structure, complete with the resolved joint loads and member referencing system is shown in Fig. 4.4.

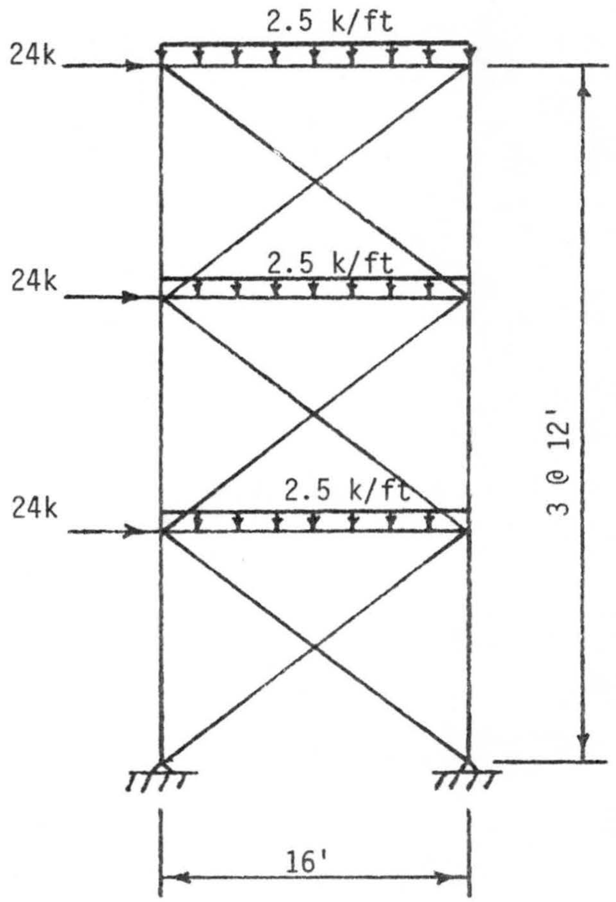


Fig. 4.3
Structural configuration and design loads.

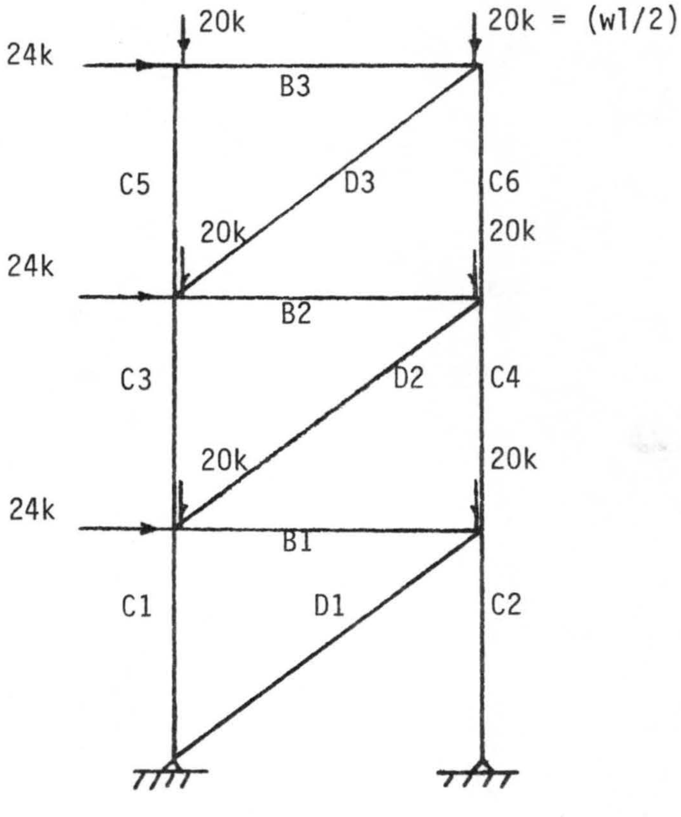


Fig. 4.4
Modeled structure with member references.

The C, B, and D member references abbreviate the type of member present, i.e. C=column, B=beam, and D=diagonal. The maximum lateral deflection specified for this structure is assumed to be 0.800 in.

The resolved member forces from both the gravity load case (dead + live) and the combined load case, 3/4(dead + live + wind), in addition to the maximum member design loads, are shown in Fig. 4.5. Important items to note concerning the analysis are:

1) the member tensile forces are assigned as positive quantities, while the compressive forces are assigned as negative quantities, and
2) it is assumed that the full wind force may act in either direction. Therefore, each opposing column effective in the braced frame must be the same as its symmetric counterpart, and must be designed on the basis of the maximum anticipated load in either member.

The results of a very approximate member design are shown in Fig. 4.6. Member areas were assigned based on a very simplified sizing algorithm including the member type (column, beam, or diagonal), the type of force experienced by the member (tensile or compressive), and on the magnitude of the force applied:

Columns and Diagonals: Compressive forces/15 ksi
Tensile forces/20 ksi

Beams: (Compressive forces/15 ksi) +
$3 in^2$ (allowance for bending)

The unit loading system and the resolved member forces are shown in Fig. 4.7. The deflection calculations, including the total structure deflection and the member deflection influence parameters,

Member	(D+L) k	(W) k	3/4(D+L+W)k	Design load
C1	-60	+54	-------	-126.0
C2	-60	-108	-126.0	-126.0
C3	-40	+18	-------	-70.5
C4	-40	-54	-70.5	-70.5
C5	-20	0	-------	-28.5
C6	-20	-18	-28.5	-28.5
B1	0	-72	-54.0	-54.0
B2	0	-48	-26.0	-26.0
B3	0	-24	-18.0	-18.0
D1	0	+90	+67.5	+67.5
D2	0	+60	+45.5	+45.5
D3	0	+30	+22.5	+22.5

Fig. 4.5

Member design loads.

Member	Design load (k)	Area (in^2)
C1 & C2	-126.0	8.40
C3 & C4	-70.5	4.70
C5 & C6	-28.5	1.90
B1	-54.0	6.60
B2	-36.0	5.40
B3	-18.0	4.20
C1	+67.5	3.375
C2	+45.0	2.250
C3	+22.5	1.125

Fig. 4.6

Computed member areas.

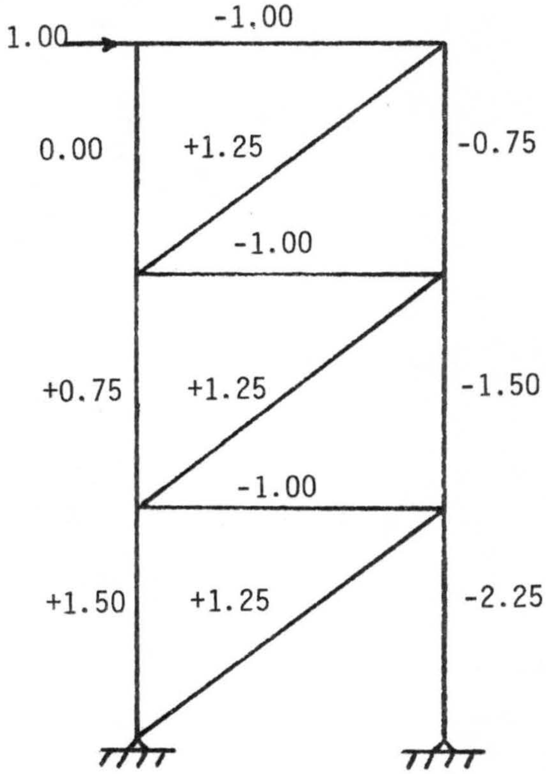


Fig. 4.7
Unit loading system.

are shown in Fig. 4.8. Important items to note concerning the information presented in this figure are:

1) a common modulus of elasticity, E, of 29000 ksi has been assumed, and is therefore not included in the member deflection contributions or deflection influence parameters,

2) the specified maximum allowable deflection of 0.800 in, or H/540, is greatly exceeded (the computed structure drift is 1.323 in, or H/327; 65.4% greater than that desired), and

3) since the columns C1 and C2, C3 and C4, and C5 and C6 must retain equal areas during and following the member area modification procedure, their respective member deflection influence parameters must be grouped together and averaged (see Fig. 4.9).

The deflection influence parameters are plotted for comparison purposes in Fig. 4.10. It is important to note that this plot is unique for this particular braced frame and is dependent on the structure geometry, design loads, and approximations and assumptions employed in the structural analysis and design.

The results of the optimization routine are presented in Fig. 4.11. Following four successive decreases, the maximum allowable deflection exceeds the computed lateral drift. The procedure necessary to achieve the exact deflection limit, and the final structure deflection check, are shown in Fig. 4.12. The final member areas are presented in Fig. 4.13. The number of significant digits in these calculations has purposely been extended beyond that necessary for most hand computations.

Member	N (k)	n	L(in)	$A(in^2)$	$\frac{NnL}{A}$ *	$\frac{Nn}{A^2}$ *
C1	+54	+1.50	144	8.4	1388.571	1.148
C2	-108	-2.35	144	8.4	4165.714	3.444
C3	+18	+0.75	144	4.7	413.617	0.611
C4	-54	-1.50	144	4.7	2481.702	3.667
C5	0	0.00	144	1.9	0.000	0.000
C6	-18	-0.75	144	1.9	1023.158	3.740
B1	-72	-1.00	192	6.60	2094.545	1.653
B2	-48	-1.00	192	5.40	1706.667	1.646
B3	-24	-1.00	192	4.20	1097.143	1.361
D1	+90	+1.25	240	3.375	8000.000	9.977
D2	+60	+1.25	240	2.250	8000.000	14.815
D3	+30	+1.25	240	1.125	8000.000	29.630

* common E = 29000ksi

$$\text{Structure deflection} = \frac{38371.118}{29000.000} = 1.323 \text{ in}$$

Fig. 4.8
Deflection calculations.

Members	$\frac{Nn}{A^2}$
C1 & C2	2.296
C3 & C4	2.139
C5 & C6	1.870

Fig. 4.9
Grouped member
deflection influence parameters.

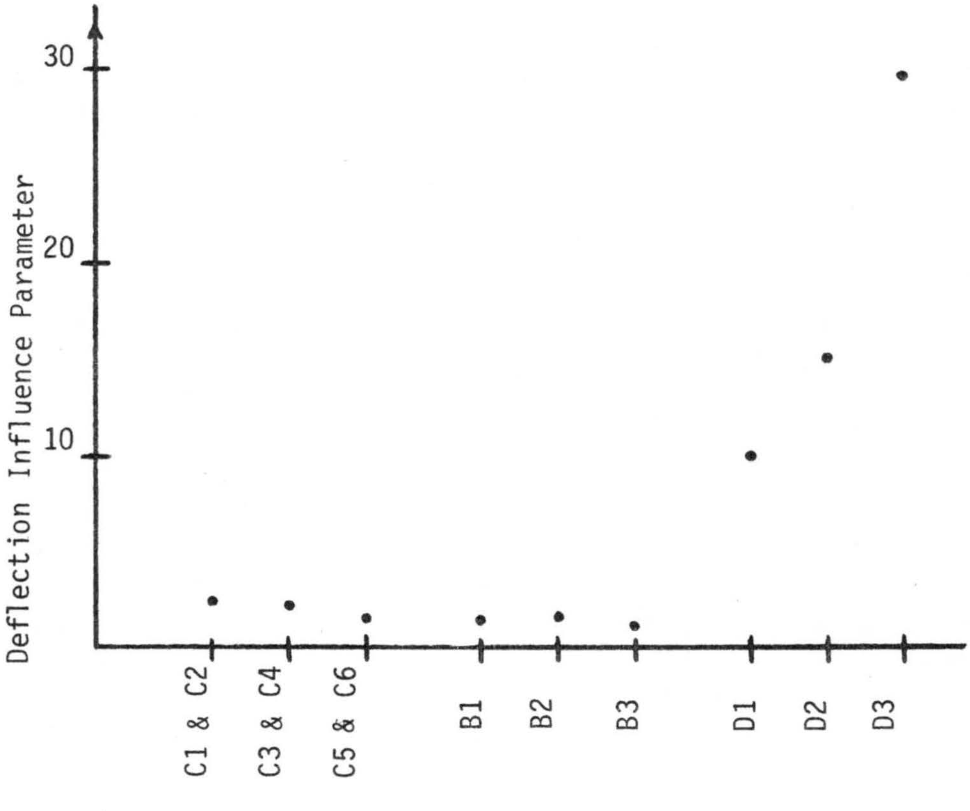


Fig. 4.10
Deflection influence parameters.

Member	N(k)	n	L(in)	$A(in^2)$	$\frac{NnL}{A}$	$\frac{Nn}{A^2}$
D3	+30	+1.25	240	1.591	5656.854	14.815

Revised structure deflection = 1.242 in.

(a)

Member	N(k)	n	L(in)	$A(in^2)$	$\frac{NnL}{A}$	$\frac{Nn}{A^2}$
D2	+60	+1.25	240	2.756	6532.083	9.877
D3	+30	+1.25	240	1.949	4618.880	9.877

Revised structure deflection = 1.156 in.

(b)

Fig. 4.11 Deflection influence parameter reductions; (a), (b), (c), and (d) represent the first through fourth reductions respectively. Following the fourth reduction, the maximum desired deflection exceeds that computed.

Member	N(k)	n	L(in)	A(in^2)	$\frac{NnL}{A}$	$\frac{Nn}{A^2}$
D1	+90	+1.25	240	7.000	3857.122	2.296
D2	+60	+1.25	240	5.715	3149.380	2.296
D3	+30	+1.25	240	4.041	2226.948	2.296

Revised structure deflection = 0.814 in.

(c)

Member	N(k)	n	L(in)	A(in^2)	$\frac{NnL}{A}$	$\frac{Nn}{A^2}$
C1	+54	+1.50	144	8.703	1340.256	1.069
C2	-108	-2.25	144	8.703	4020.767	3.208
D1	+90	+1.25	240	7.252	3722.913	2.139
D2	+60	+1.25	240	5.921	3039.797	2.139
D3	+30	+1.25	240	4.187	2149.461	2.139

Revised structure deflection = 0.796 in.

(d)

Fig. 4.11 con't.

Eq. 4.3 $\Delta r = 15171.118/E$ in.

Eq. 4.4 $\Delta m = 29554.285/E$ in.

Eq. 4.5 $\Delta n = 14383.167/E$ in.

Eq. 4.6 $\Delta c = 14273.194/E$ in.

Eq. 4.7 ADF = 0.992

Member	L(k)	n	L(in)	A(in^2)	$\frac{NnL}{A}$	$\frac{Nn}{A^2}$
C1	+54	+1.50	144	8.636	1350.554	1.086
C2	-108	-2.25	144	8.636	4051.661	3.258
D1	+90	+1.25	240	7.197	3751.797	2.172
D2	+60	+1.25	240	5.876	3063.450	2.172
D3	+30	+1.25	240	4.155	2166.072	2.172

Fig. 4.12 Procedure to achieve the maximum allowable deflection.

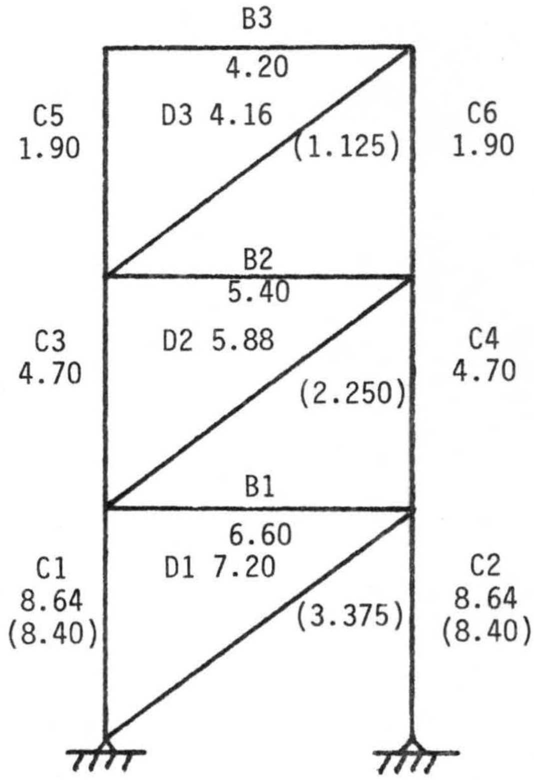


Fig. 4.13

Final member areas.

Original areas are shown in parentheses for comparison purposes.

To best realize the effectiveness of this optimization routine, the following important values are considered:

Original structural steel volume	=	10670.40 in^3
Final structural steel volume	=	15774.72 in^3
Structural steel volume increase	=	47.84%
Original structure deflection	=	1.323 in
Final structure deflection	=	0.800 in
Structure deflection decrease	=	39.53%
Structural stiffness increase	=	65.38%

Note the sizeable increase in structural stiffness (65.38%) and subsequent decrease in structure deflection provided by the smaller percentage increase (47.84%) in structural steel.

The problem, as presented above, does not recognize that counters must be placed in both directions if the wind can act from either direction. The presence of counter members can be modeled by including a zero force diagonal member in each panel with an area set equal to the active tensile member. For this case, the information in Figures 4.4 through 4.7 and Fig. 4.9 remains valid. The deflection influence parameters for the diagonals in Figures 4.8 and 4.10 are reduced by fifty percent with the inclusion of the three inactive diagonals. An optimization procedure similar to that presented in Figures 4.11 and 4.12 resulted in the final member areas shown in Fig. 4.14. The summary of the design values for this structure are:

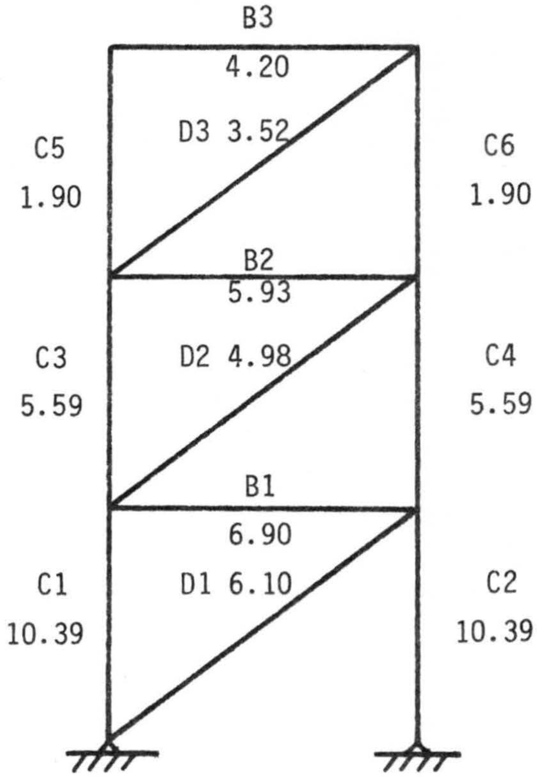


Figure 4.14
Final member areas.

Original structural steel volume	=	10670.40 in^3
Final structural steel volume	=	15475.49 in^3
Structural steel volume increase	=	45.03%
Structural deflection decrease	=	39.53%
Structural stiffness increase	=	65.38%

A comparison of the two solutions shows that less additional area is assigned to the diagonals of the structure (for example, 6.10 in^2 versus 7.20 in^2 for the lowest level), and more to the columns (for example, 10.35 in^2 versus 8.64 in^2 for the lowest columns), when the areas of the inactive diagonals are recognized in the optimization procedure.

4.5 Program WTD1 and Additional Design Examples

Program WTD1 (Wind Truss Design 1) is a computerized structural analysis and design routine specifically written to perform the deflection influence parameter optimization procedure, as presented in Sections 4.2 and 4.3, for Type 2 braced frames which exhibit excessive lateral sway when subjected to the full design wind loads. The program requires the user to develop a structure data file, complete with joint and member information, loading data, and deflection criteria. The program computes the total structure drift, compares this value versus the maximum permissible deflection, and performs the optimization procedure if necessary. The results of the program and the optimization routine are in terms of the revised member areas.

A documentation and users manual for Program WTD1 is located in Appendix A. The program's operating sequence and program listing are presented in Appendix B. Finally, three specific design examples, incorporating Program WTD1, are shown in Appendix C.

Design Example 1 involves the design and optimization of a one-bay three-story determinate truss. This problem was selected primarily to demonstrate the use of Program WTD1 for drift computation and member area modification.

Design Example 2 entails the design and optimization of a one-bay eight story indeterminate truss. This braced frame was selected to demonstrate more completely the design capabilities offered by the program.

Design Example 3 involves the optimization of a one-bay eight-story K-braced frame. Initially designed for gravity loads only, this truss is first optimized by Program WTD2, and then checked for member adequacy using the analysis results from a modified version of the original program.

CHAPTER V

SUMMARY AND RESEARCH NEEDS

As demonstrated in the previous chapter, the deflection influence parameter optimization procedure permits the designer to simply, efficiently, and systematically design Type 2 braced framing systems to satisfy structure stiffness requirements. The deflection influence parameter, defined as the member deflection contribution divided by the member volume, permits the designer to rapidly identify those members least efficient in providing the stiffness necessary to limit excessive lateral drift. The optimization routine, as presented, has the potential to become an effective and powerful design tool. The optimization method itself is simple to understand and does not require a knowledge of formal optimization procedures.

Conceptually, this procedure could be extended to rigid framing systems by the inclusion of member deflection contributions due to bending effects. For such a rigid frame optimization routine, both member areas and moments of inertias would become subject to modification. This would require a more sophisticated and involved optimization method and computer program.

Parameter studies involving the concepts of the deflection influence parameter and its relationship to the approximations and assumptions commonly employed in designing various braced framing systems remain to be examined. Studies concerning the optimization of multistory multi-bay truss arrangements, such as cap and belt trusses, also remain to be attempted.

This optimization routine could be incorporated into a design program, which for a given structural configuration and set of design loads would select standard steel sections adequate for both strength and stiffness considerations. An interactive form of the optimization program could also be developed and would offer many benefits by permitting designer participation and intervention.

In the future, as building dead weights continue to decrease and height-to-width ratios continue to increase, the attention given to lateral load design and the efficiency of lateral load resisting systems will also increase. The optimization procedure as presented in this study can become a valuable design tool for the designer seeking to produce more efficient and economical structures.

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APPENDIX A

PROGRAM WTD1

DOCUMENTATION AND USERS MANUAL

APPENDIX A

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APPENDIX A

PROGRAM WTD1

DOCUMENTATION AND USERS MANUAL

A.1 Introduction

Program WTD1 (Wind Truss Design 1) is a computerized structural analysis and design program for plane trusses composed of structural steel sections. Although the program is oriented towards optimizing member areas to achieve specified structure drift limitations, its analysis capablities may also be employed to obtain member end actions required to check gravity load cases and/or combined loading cases.

Program WTD1 operates basically as an analysis, design, and reanalysis program, modifying member areas to meet lateral deflection requirements for statically determinate or indeterminate pinned trusses with members previously sized for strength adequacy. The cycle of member area modification in the design stage followed by a reanalysis is repeated until user specified drift limitations are satisfied (see Fig. A.1). Program convergence is very rapid. Only one iteration is required for a determinate pinned truss, and usually only two iterations are needed for an indeterminate pinned truss. Program WTD1 provides the user with the optimal areas of all members as necessary to limit the structural deflection at any single point and direction to an acceptable value. Lateral deflection at the top of a building will normally be the deflection of interest.

Program WTD1 is written in ANSI FORTRAN IV computer language and has operated sucessfully on the Colorado State University CDC CYBER 172 (NOS1.1). Although this program requires a fixed format data file, it may be readily utilized by designers having little or no familiarity with computer programming. The following provides a description of the program, its capabilities, and a complete summary of the required data file.

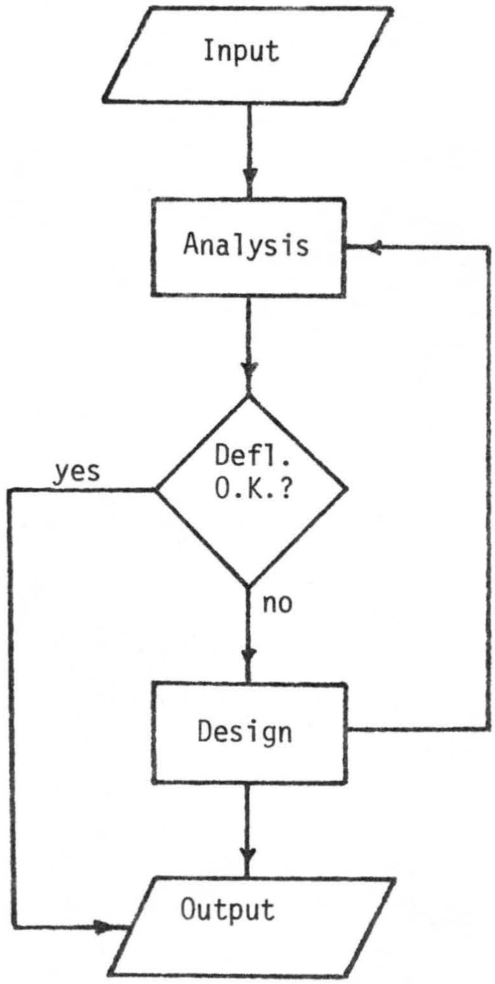

Fig. A.1 Generalized flow chart for Program WTD1.

A.2 Program Capabilities

Program WTD1 is capable of performing structural analysis for plane trusses with pinned joints and is further capable of redesigning and optimizing member sizes to satisfy lateral deflection constraints. The program assumes that the structure has been previously designed to withstand gravit loading cases and/or combined loading cases representative of those which may be expected to occur on the structure during its intended use.

The program is currently capable of optimizing trusses containing fifty (50) joints with a maximum of one-hundred (100) members. Any number of possible loading cases may be analyzed, however, due to present program core restrictions, only one at a time is allowed.

A.3 Required Input

Program WTD1 requires the user to prepare a data file specifying the following information:

(1) Structure data, including

- (a) the title of the structure
- (b) the number of joints and members in the structure
- (c) joint coordinates and support conditions
- (d) member incidences and corresponding member properties

(2) Loading data, including

- (a) the number of applied joint loads
- (b) the location, direction and magnitude of each applied joint load
- (c) the location and direction of the applied unit load

(3) Deflection criteria, including

(a) the maximum allowable deflection

(b) the deflection convergence limit

(c) the structure type (determinate or indeterminate)

A.4 Listed Output

Program WTD1, in addition to listing all input values, will also print the following computed values for each successive iteration:

(1) member axial forces for both the real and unit load systems

(2) member deflection contributions and deflection influence parameters

(3) revised deflection computations and member areas (if the maximum allowable deflection is exceeded)

In addition, the program computes the initial and final volume of structural steel present in the truss.

A.5 Program Units

The user must insure the proper data units to receive appropriate results. At present, Program WTD1 is designed to incorporate the units of kips (K) and inches (IN). A summary of the input units includes:

Joint coordinates--IN

Member cross-sectional area--square inches (IN2)

Member modulus of elasticity--kips per square inch (KSI)

Applied joint loads--K

Applied unit load--dimensionless

Maximum allowable deflection--IN

Deflection convergence limit--IN

A summary of the output units includes:

Member length--IN

Structural steel volume--cubic inches (IN3)

Member and structure deflections--IN

Member, member group, and structure deflection influence parameters--units not specified on output listing

A.6 Coordinate Systems

Program WTD1 incorporates two coordinate systems; the first describing the structure geometry, and the second describing the member geometry. The user is free to specify any structural coornate system, while the member coordinate system is dependent upon the member and joint numbering scheme and is therefore assigned internally by the program.

In most computer programs, the capital letters X, Y, and Z are incorporated for the structural coordinate system, while the lower case x, y, and z are reserved for the member coordinate system (see Fig. A.2a and b).

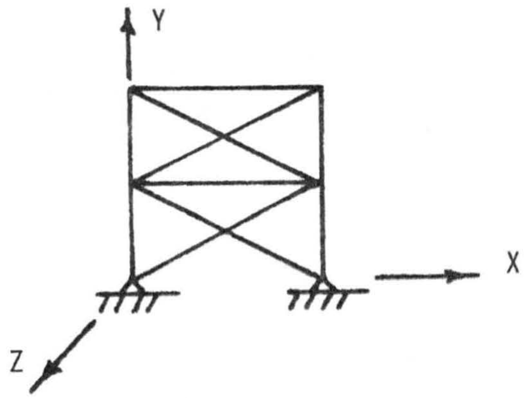

Fig. A.2a
Structural coordinate system

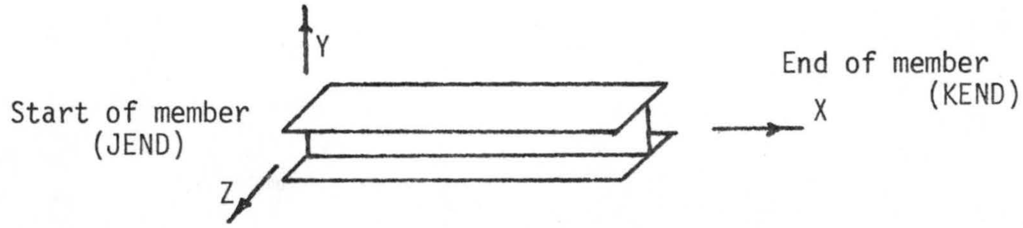

Fig. A.2b
Member coordinate system

In most instances, it is convenient to locate the structure so that all structural coordinates are positive. Since the X, Y, and Z system follows the right hand rule, the Z axis is out of the plane of the page. Therefore, all structural coordinates must lie in the X-Y plane.

A.7 Member Incidences

Each joint and member in the structure must be assigned a unique number or "identifier" selected by the user. The member incidences, or JEND and KEND joint numbers, are used to define the location of each member in the structure.

Program WTD1 requires all joints and all members to be numbered consecutively, beginning with the integer one. Due to program dimensioning, the maximum difference in JEND and KEND values for any single member is limited to nine; the user should be alert to this fact when assigning joint numbers.

If the member shown in Fig. A.2b represents the beam B-C as shown in Fig. A.3, member incidences may be defined in one of two ways. If the joints A, B, C, and D are numbered consecutively (i.e. A=1, B=2, C=3, and D=4) then beam B-C may be defined as having JEND=2 and KEND=3. Beam B-C will then be represented as shown in Fig. A.2b.

However, if the joint assignments are reversed to read JEND=3 and KEND=2, beam B-C will now be oriented in the opposite direction. The user should, if at all possible, be consistent by assigning all members in a left to right or right to left orientation. Otherwise, some of the member axial forces, although being correct in magnitude, will be reversed in sign, making the output more difficult to properly interpret.

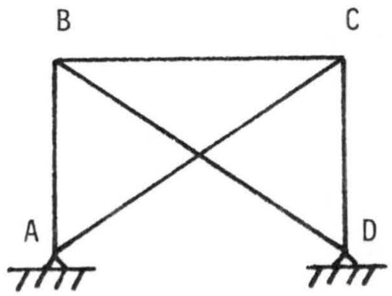

Fig. A.3
Member incidences

A.8 Statement Sequence

Program WTD1 requires a fixed format data input. It is therefore imperative that the user understand and employ the following data file sequence.

The first data card is reserved for the structure title and any other information desired by the user to identify the structure. All eighty (80) columns of the statement are read and will be printed. The information on this card is not used by the program and there are no restrictions on its form.

The second statement must list, in integer format, the number of joints and members in the structure. The number of joints must be located in the first 5 columns and the number of members must be in the second 5 columns. Note that all integer values must be right justified, i.e. if the structure has seven (7) joints and twelve (12) members, the 7 must be placed in column 5 and the 12 must be placed in columns 9 and 10.

Next, for each joint in the structure, a statement is required specifying the following information:

Joint number (an integer value, columns 1-5, right justified)

X and Y coordinates (fixed point numbers, columns 6-15 and 16-25 respectively)

Joint support conditions in the X and Y directions (integer values, zero (0) if the joint displacement is free in the direction being considered, one (1) if the joint location is fixed, appropriate values placed in columns 30 and 35 for X and Y directions, respectively)

For each member in the structure a statement is required specifying the following information:

Member number (an integer, columns 1-5, right justified)

Member JEND and KEND values (integers, columns 6-10 and 11-15 respectively, right justified)

Member area (fixed point number, columns 16-25)

Member modulus of elasticity (fixed point number, columns 26-35)

Member IDENT value (integer, columns 36-40, right justified)

The member IDENT feature allows the user to specify certain members or groups of members which must possess equivalent cross sectional areas should one or more of the member areas be affected by the deflection influence parameter design routine. Each member in the same IDENT member group must share the same IDENT value beginning with the integer one (1), and continuing consecutively for as many IDENT members or groups of members are present in the structure.

Following the last member statement, a card indicating the number of applied joint loads is required (an integer, columns 1-5, right justified). For each joint load or combinations of joint loads (i.e. an X force and Y force at the same joint) a statement is required specifying the following information:

Joint number (an integer, columns 1-5, right justified)

Joint loads, X force and/or Y force (fixed point numbers, X and Y forces placed in columns 6-25 and 16-26 respectively)

Following the last joint load card describing the applied joint loads, another card with the same format must be included to specify the location and direction of the unit load. The magnitude of the unit load must be given, even though its value is known to be 1.00.

The final data statement must provide the program with the following deflection information:

Maximum allowable deflection (fixed point number, columns 1-10)

Deflection convergence limit (fixed point number, columns 11-20)

Structure type, zero (0) if statically determinate, one (1) if statically indeterminate (an integer value, columns 20-25, right justified).

The deflection convergence limit is the maximum tolerance above the allowable deflection permitted by the designer. For a determinate truss a convergence limit is not required; the design procedure will force the structure to achieve the exact maximum allowable deflection. For an indeterminate structure, deflection convergence limits of up to 0.001 in. seem reasonable; the analysis check usually results in a structural deflection at or slightly below the desired maximum allowable deflection.

This concludes the structure data file. A simplified diagram indicating the appropriate statement sequence and format specifications is shown in Fig. A.4. A listing of the input cards required to run Design Example 1 may be seen in Appendix C.1. Appendix B discusses the program operating procedure, complete with a detailed flow diagram and a listing of Program WTD1.

A.9 Debugging Hints

Possible data file errors may be eliminated by carefully editing the data deck for:

(1) Keypunching errors--proper type (integer or fixed values), proper location on statement

(2) Missing cards--remember to include a title statement, all joint and member statements, and all load data

(3) The maximum JEND-KEND specification (nine or less)

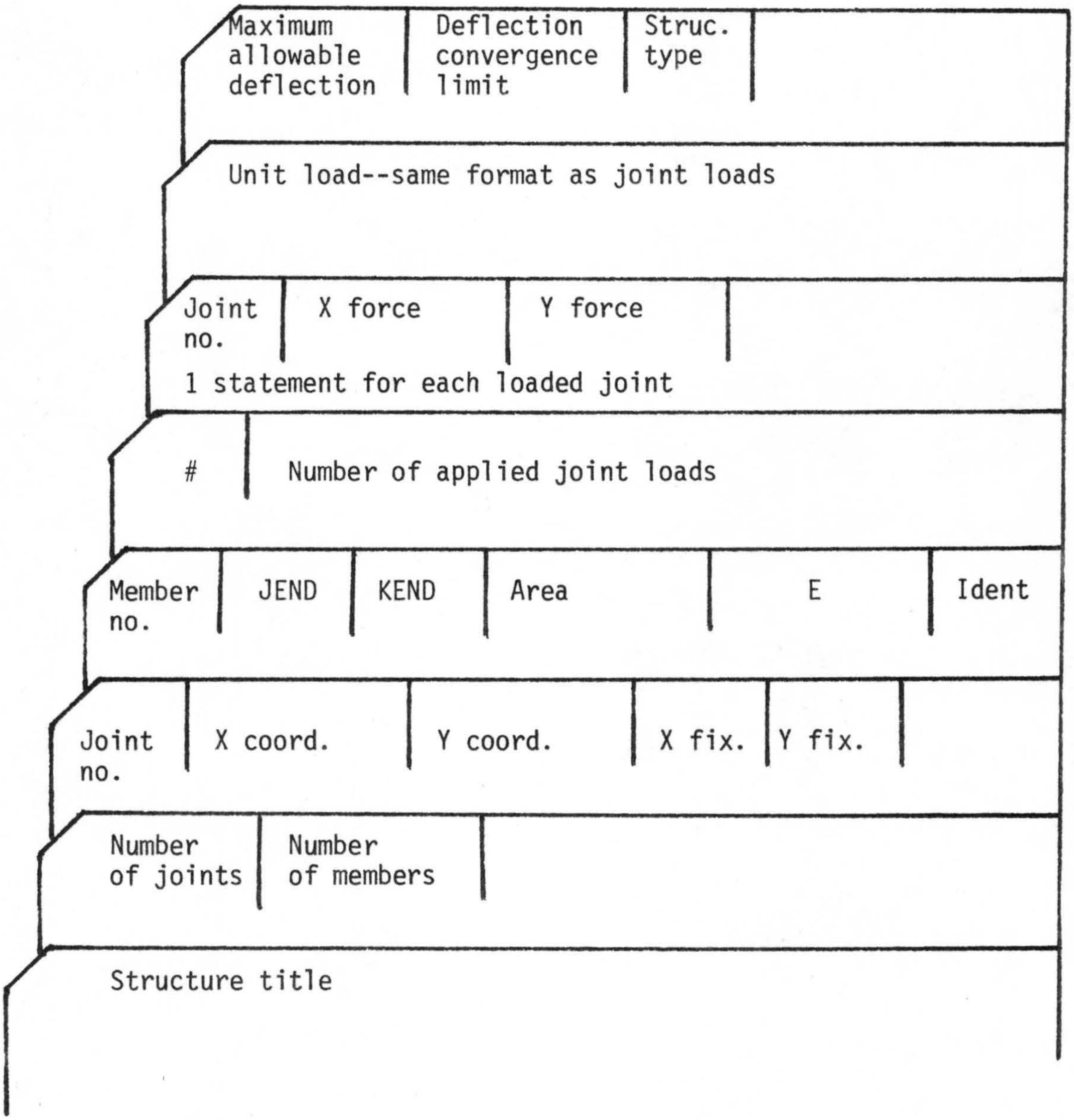

Fig. A.4
Structure data file

APPENDIX B

PROGRAM WTD1

OPERATING SEQUENCE AND

PROGRAM LISTING

APPENDIX B

PROGRAM WTD1

OPERATING SEQUENCE AND PROGRAM LISTING

B.1 Introduction

This appendix provides a detailed examination of Program WTD1, including its operating procedure and flow of control. To aid in this discussion, an itemized flow diagram of the program is shown in Fig. B.1. Each symbol on the flow chart is individually discussed, in addition to specialized program functions and features. A complete listing of the program, together with a discussion of possible program revisions, concludes this appendix.

B.2 Read Structure Data

The first segment of Program WTD1 defines and reserves array core space, in addition to reading, storing, and printing the structure data. Structure data includes the structure title, the number of joints and members present, and an individual statement for each joint and member in the structure (see Appendix A.8 Statement Sequence).

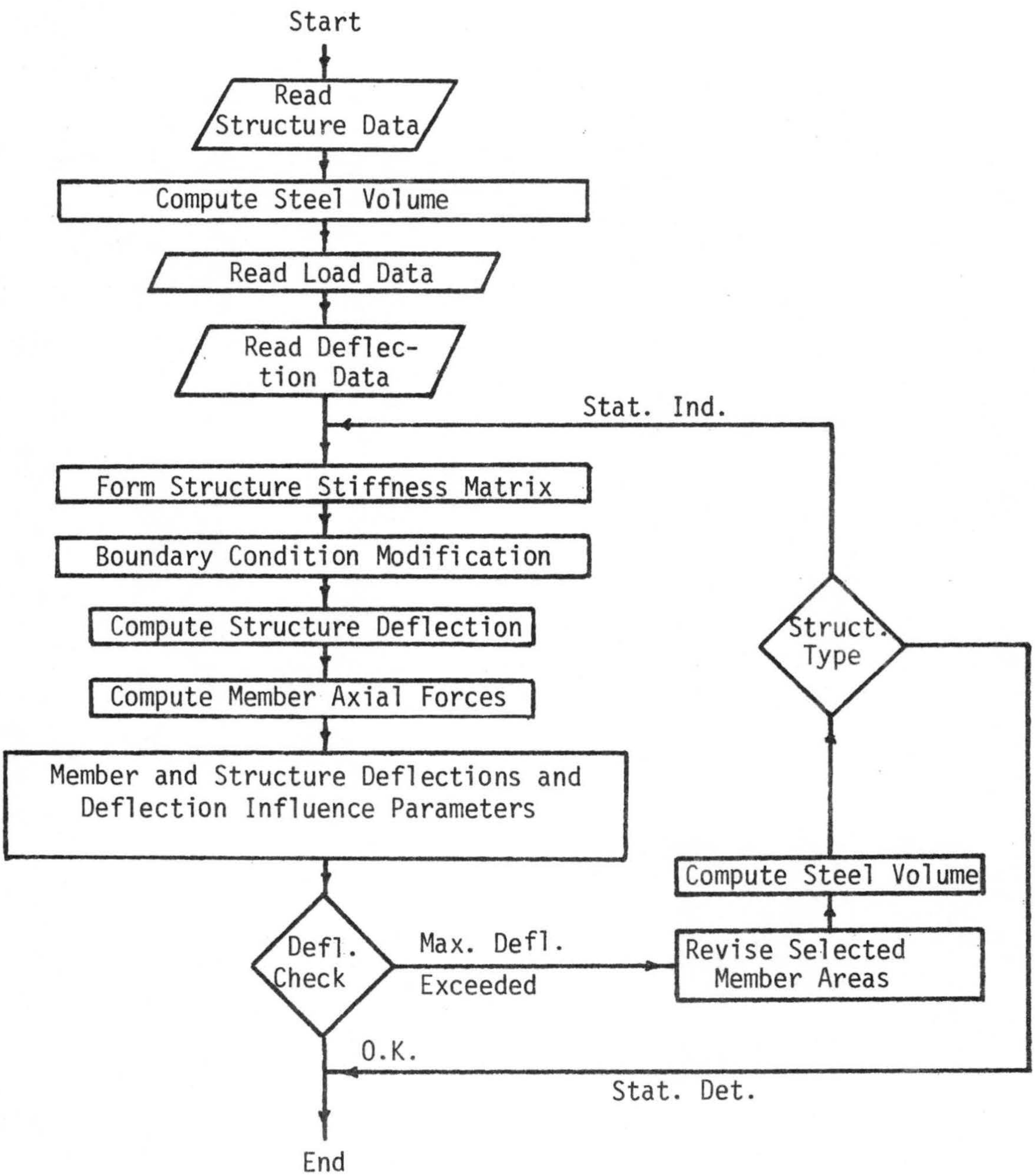

Fig. B.1
Program WTD1
Flow Diagram

Important storage locations in this segment of the program include:

ITITLE	Structure title and/or other user desired information (no format restrictions).
NJOINT and NMEM	Total number of joints and members in the structure.
JN	Individual joint number.
X(JN) and Y(JN)	X and Y coordinates defining joint JN.
XTFIX(JN) and YTFIX(JN)	X and Y joint fixity conditions (see Appendix A.8).
M	Individual member number.
JC(M) and KC(M)	Member JEND and KEND values (see Appendix A.7).
AA(M)	Member cross-sectional area.
E(M)	Member modulus of elasticity.
IDENT(M)	Member IDENT value (see Appendix A.8).
ALEN(M)	Computed member length (for use as a data file check).
NUMID	Number of IDENT member groups.

B.3 Compute Steel Volume

Program WTD1 computes and prints the initial and final volumes of structural steel present in the wind truss. The initial volume, calculated from the computed member lengths and input member areas, is the quantity of steel required to insure strength adequacy. The final volume, incorporating the design revised member areas, is the quantity of steel required to provide adequate strength as well as limit excessive lateral drift. These two values permit the designer to estimate the wind premium, the additional steel required (above that necessary to sustain gravity loads) to resist lateral loads and prevent excessive lateral deflection (see Chapter 3).

B.4 Read Load Data

Segment three of Program WTD1 reads, stores, and prints the structure loading data for both the real and the unit loading systems. Load data includes the number of applied joint loads acting on the structure, an individual statement for each joint load or combination of joint loads, and a card describing the applied unit load (see Appendix A.8).

Important storage locations in this segment of the program include:

NJTL	Number of applied joint loads.
JTL(4)	Initial joint load storage.
AJ(100,2)	Actions at joints matrix.
PA(100,2)	Permanent AJ storage.

The AJ matrix, which is rewritten during the Gaussian elimination procedure (see Appendix B.8), must be duplicated and saved in the event that a second analysis iteration is required. Actually, this AJ=PA duplication is necessary only for statically indeterminate structures; determinate structures are solved in one iteration.

B.5 Read Deflection Criterion

The fourth segment of Program WTD1 reads, stores, and prints the user specified maximum allowable deflection and the deflection convergence limit (see Appendix A.8). The user must also indicate the structure type: statically determinate or indeterminate. If the structure is designated as statically determinate, the program will terminate following one complete analysis-design iteration. If the structure is specified as statically indeterminate, an analysis check for the revised structure will be performed, in addition to further iterations should the maximum allowable drift be exceeded.

Important storage locations in this segment of the program include:

DMAX	Maximum allowable deflection.
EPS	Deflection convergence limit.
DMAXH	Maximum possible deflection (DMAX + EPS).
NTYPE	Structure type, statically determinate or indeterminate.

B.6 Form Structure Stiffness Matrix

The fifth segment of Program WTD1 develops the individual member stiffness matrices, rotates them into the structural coordinate system using direction cosine matrices, and ultimately creates the structure stiffness matrix. Four subroutines, or subprograms, invoked by the main program are incorporated into this procedure:

SUBROUTINE FORMRT	Develops the individual member rotation matrices (commonly termed "direction cosine" matrices) used to relate the member

coordinate system to the structure coordinate system. FORMRT also creates the transposed member rotation matrices.

SUBROUTINE FORMSM	Develops the individual member stiffness matrices, incorporating the AE/L axial stiffness terms.
SUBROUTINE MULT	A general purpose matrix multiplication routine, used in this segment of the program to create the rotated member stiffness matrices.
SUBROUTINE INDEX	Develops the member-structure referencing system with the member JEND and KEND values.

The structure stiffness matrix, composed of the individual rotated member stiffness matrices, is stored in a triangularized pattern with the main diagonal placed in the first column of the matrix. This is possible since the rotated member stiffness matrices are always symmetrical about their main diagonal. Due to program dimensioning, any number with a JEND-KEND difference greater than nine will exceed the maximum specified bandwidth (IBAND) of the matrix (see Appendix A.7).

Important storage locations in this segment of the program include:

COST(M) and SINT(M)	Member direction cosine values (member orientation with respect to the structural coordinate system).
RT (4,4)	Member rotation (or direction cosine) matrices.

RTRANS (4,4)	Transposed member rotation matrices.
SM (4,4)	Member stiffness matrices.
IN (4)	Member-structure referencing coordinates.
TEMP (4,4)	Temporary storage location used during matrix manipulations and multiplications.
IBAND	Maximum computed stiffness matrix bandwidth: IBAND = 2 x [Max. (JEND-KEND) + 1]
S	Structure stiffness matrix.

B.7 Boundary Condition Modifications

Segment six of Program WTD1 modifies the structure stiffness matrix and the actions at joints matrix for known joint support conditions. Each joint in a plane truss may have two structural translations; one in the structure X direction, and one in the structure Y direction. Since the stiffness method of structural analysis solves for structural joint displacements, or mathematically

$$S \times DJ = AJ$$

where S = Structure stiffness matrix,

DJ = Displacements at joints matrix,

and AJ = Actions at joints matrix,

zero joint displacements are accomplished by first replacing the appropriate diagonal term of the S matrix (determined by the XTFIX and YTFIX values) with a large value, 1×10^{60}, while zeroing the corresponding AJ term. For all practical purposes, the resulting joint displacement is zero.

B.8 Compute Structure Deflections

The structural deflections are obtained by a Gaussian elimination procedure contained in Subroutine SOLVE. SOLVE systematically eliminates individual rows of the S and AJ matrices, and back substitutes for the displacements at joints or DJ matrix. During the elimination process, the contents of the original S and AJ matrices are effectively destroyed. If a second analysis iteration is required, the S matrix must be recreated incorporating the revised member areas, while the AJ matrix must be rebuilt from a permanent actions at joints matrix, PA (see Appendix B.4).

B.9 Compute Member Axial Forces

In this segment of Program WTD1, member end actions for both the real and the unit loading systems are determined from the structure joint displacements. This procedure involves rotating the appropriate joint displacements into the individual member coordinate systems, then multiplying by the appropriate member stiffness matrices. Both member axial and shear effects are computed and stored; however, for the remainder of the program, only member axial forces are considered. Member shear actions are included for the designer wishing to modify the program by incorporating transverse member loading conditions.

Important storage locations in this segment of the program include:

AM(400,2)	Member end actions, including axial and shear effects for both the real and the unit loading systems.

RNN(100) and RN(100)	Member axial forces for the real and the unit loading systems respectively.

B.10 Member and Structure Deflections and Deflection Influence Parameters

Program WTD1 uses two subroutines, DEFL and GROUP, to compute and store the member and member IDENT deflection contributions and deflection influence parameters, in addition to the overall structure deflection and deflection influence parameter.

Important storage locations for this segment of the program include:

DEF(100)	Member deflection contributions.
DEFP(100)	Member deflection influence parameters.
DEFPG(100)	Member group deflection influence parameters.
DEFS	Structure deflection (at point of unit load).
DEFPS	Structure deflection influence parameter.

B.11 Deflection Check

Following the structure deflection calculation, Program WTD1 compares this value, DEFS, with the maximum possible deflection, DMAXH. If the computed deflection is less than or equal to DMAXH, the program is terminated. If the computed deflection exceeds DMAXH, program control is transferred to the design phase.

B.12 Revise Selected Member Areas

The first operation performed by the program in the design phase involves ordering the member and member group deflection influence parameters in a highest to lowest array. The program then proceeds to select successively lower deflection influence parameters, redesigning the members or member groups involved, and recomputing the structure deflection. The program continues in this manner until the user specified maximum allowable drift exceeds or equals the computed value. The maximum deflection influence parameter (DEFIP) required to achieve the allowable drift is computed and used by the program to calculate the revised member and/or member group areas. If the structure is statically determinate, the analysis-design procedure is complete at this point and the program is terminated. If the structure is indeterminate, an analysis is required to confirm the revised design.

Important storage locations for this segment of the program include:

H(100)	Ordered deflection influence parameters.
RA(100)	Revised member cross sectional areas.
T(100)	Temporary storage array used in ordering the deflection influence

	parameters and in revising the individual member areas.
NTR	Counter indicating the current value of the maximum deflection influence parameter (MDEFP).
DEFC	Structure deflection due to members or member groups with revised areas.
DEFNC	Structure deflection due to members retaining their original areas.
MDEFP	Maximum deflection influence parameter.
DEFN	Maximum additional structure deflection.
DEFIP	Largest computed member deflection influence parameter.

B.13 Structure Type

Following the design sequence, the program performs a structure type (NTYPE) test. If the structure is statically determinate, the design is complete and the program is terminated. If the structure is statically indeterminate, the program will form a new structure stiffness matrix using the revised member areas, and be recycled through another analysis phase. If, following the reanalysis, the computed structure deflection still exceeds the maximum possible deflection, the program will perform another design-analysis iteration.

B.14 Program Listing

A complete listing of Program WTD1 follows. It should be noted that the program capabilities and features, as presented and discussed in Appendices A and B, may be easily modified to accomodate the individual needs of the designer. Assistance concerning appropriate storage location revision is offered following the program listing. Appendix C presents a number of design examples utilizing Program WTD1.

Listing of Program WTD1

Read structure data

```
      PROGRAM WTD1 (INPUT,OUTPUT,TAPE1=INPUT,TAPE2=OUTPUT)
      COMMON /INFORMA/ RT(4,4),SINT(100),COST(100),RTRANS(4,4)
      COMMON /INFORMB/ ALEN(100),E(100),AA(100),SM(4,4)
      COMMON /INFORMC/ JC(50),KC(50),IN(4)
      COMMON /INFORMD/ S(100,20),AJ(100,2),DJ(100,2)
      COMMON /INFORME/ RNN(100),RN(100),DEF(100),DEFP(100),DEFPG(100)
      COMMON /INFORMF/ IDENT(100),TVOL(100),TDEF(100),AVDEFP(100)
      DIMENSION ITITLE(8), X(50), Y(50), XTFIX(50), YTFIX(50)
      DIMENSION TEMP(4,4), SMR(4,4), AM(400,2), RA(100)
      DIMENSION JTL(4), D(4), AMT(4), H(100), T(100), PA(100,2)
      REAL LX,LY,MDEFP,JTL
      INTEGER XTFIX,YTFIX
C
C     STRUCTURE DATA
C
      READ (1,148) ITITLE
      WRITE (2,149) ITITLE
      READ (1,150) NJOINT,NMEM
      WRITE (2,151) NJOINT
      WRITE (2,152) NMEM
      DO 101 I=1,NJOINT
         READ (1,153) JN,X(JN),Y(JN),XTFIX(JN),YTFIX(JN)
  101 CONTINUE
      WRITE (2,154)
      DO 102 I=1,NJOINT
         WRITE (2,155) I,X(I),Y(I),XTFIX(I),YTFIX(I)
  102 CONTINUE
      DO 103 I=1,NMEM
         READ (1,156) M,JC(M),KC(M),AA(M),E(M),IDENT(M)
  103 CONTINUE
      WRITE (2,157)
```

Compute steel volume

```
      NUMID=0
      VOLUME=0.0
      DO 104 I=1,NMEM
         JJ=JC(I)
         KK=KC(I)
         LX=X(KK)-X(JJ)
         LY=Y(KK)-Y(JJ)
         ALEN(I)=SQRT(LX*LX+LY*LY)
         WRITE (2,158) I,JC(I),KC(I),ALEN(I),AA(I),E(I),IDENT(I)
         COST(I)=LX/ALEN(I)
         SINT(I)=LY/ALEN(I)
         IF (IDENT(I).GT.NUMID) NUMID=IDENT(I)
         VOLUME=VOLUME+(AA(I)*ALEN(I))
  104 CONTINUE
      WRITE (2,159) VOLUME
```

```
C
C     LOADING DATA
C     APPLIED JOINT LOADS
C
      N2=2*NJOINT
      DO 106 I=1,2
         DO 105 J=1,N2
            AJ(J,I)=0.0
            PA(J,I)=0.0
  105    CONTINUE
  106 CONTINUE
      READ (1,160) NJTL
      WRITE (2,161)
      WRITE (2,162)
      DO 108 I=1,NJTL
```

Read load data

```
      READ (1,163) JN,(JTL(J),J=1,2)                                    A 620
      WRITE (2,164) JN,(JTL(J),J=1,2)                                   A 630
      J2=2*JN                                                           A 640
      IN(1)=J2-1                                                        A 650
      IN(2)=J2                                                          A 660
      DO 107 IA=1,2                                                     A 670
         MA=IN(IA)                                                      A 680
         AJ(MA,1)=AJ(MA,1)+JTL(IA)                                      A 690
         PA(MA,1)=AJ(MA,1)                                              A 700
  107    CONTINUE                                                       A 710
  108 CONTINUE                                                          A 720
C                                                                       A 730
C     UNIT LOAD                                                         A 740
C                                                                       A 750
      WRITE (2,165)                                                     A 760
      WRITE (2,166)                                                     A 770
      READ (1,163) JN,(JTL(J),J=1,2)                                    A 780
      WRITE (2,164) JN,(JTL(J),J=1,2)                                   A 790
      JN=2*JN                                                           A 800
      IN(1)=J2-1                                                        A 810
      IN(2)=J2                                                          A 820
      DO 109 IA=1,2                                                     A 830
         MA=IN(IA)                                                      A 840
         AJ(MA,2)=AJ(MA,2)+JTL(IA)                                      A 850
         PA(MA,2)=AJ(MA,2)                                              A 860
  109 CONTINUE                                                          A 870
```

Read deflection criterion

```
C                                                                       A 880
C     DEFLECTION CRITERIA                                               A 890
C                                                                       A 900
      READ (1,167) DMAX,EPS,NTYPE                                       A 910
      WRITE (2,168) DMAX                                                A 920
      WRITE (2,169) EPS                                                 A 930
      DMAXH=DMAX+EPS                                                    A 940
      IF (NTYPE.EQ.0) GO TO 110                                         A 950
      WRITE (2,170)                                                     A 960
      GO TO 111                                                         A 970
  110 WRITE (2,171)                                                     A 980
  111 CONTINUE                                                          A 990
```

Form structure stiffness matrix

```
C                                                                       A 1000
C     MEMBER AND STRUCTURE STIFFNESS MATRICES                           A 1010
C                                                                       A 1020
      IBAND=0                                                           A 1030
      DO 113 I=1,N2                                                     A 1040
         DO 112 J=1,20                                                  A 1050
            S(I,J)=0.0                                                  A 1060
  112    CONTINUE                                                       A 1070
  113 CONTINUE                                                          A 1080
      DO 116 I=1,NMEM                                                   A 1090
         CALL FORMRT (I)                                                A 1100
         CALL FORMSM (I)                                                A 1110
         CALL MULT (RTRANS,4,4,SM,4,4,TEMP)                             A 1120
         CALL MULT (TEMP,4,4,RT,4,4,SMR)                                A 1130
C                                                                       A 1140
C     STIFFNESS MATRIX STORAGE                                          A 1150
C                                                                       A 1160
         CALL INDEX (I)                                                 A 1170
         DO 115 J=1,4                                                   A 1180
            JJ=IN(J)                                                    A 1190
            DO 114 K=1,4                                                A 1200
               KK=IN(K)                                                 A 1210
               IF (JJ.GT.KK) GO TO 114                                  A 1220
```

```
          KK=KK-JJ+1
          IF (KK.GT.IBAND.AND.SMR(J,J).NE.0.0) IBAND=KK
          S(JJ,KK)=S(JJ,KK)+SMR(J,K)
  114     CONTINUE
  115   CONTINUE
  116 CONTINUE
      N4=4*NMEM
      DO 119 I=1,2
        DO 117 J=1,N2
          AJ(J,I)=PA(J,I)
  117   CONTINUE
        DO 118 J=1,N4
          AM(J,I)=0.0
  118   CONTINUE
  119 CONTINUE
```

Boundary condition modifications

```
C
C     S AND AJ BOUNDARY CONDITION MODIFICATIONS
C
      RFAC=1.0E+60
      DO 123 I=1,NJOINT
        I2=2*I
        IF (YTFIX(I).EQ.0) GO TO 121
        S(I2,1)=RFAC
        DO 120 J=1,2
          AJ(I2,J)=0.0
  120   CONTINUE
  121   IF (XTFIX(I).EQ.0) GO TO 123
        I1=I2-1
        S(I1,1)=RFAC
        DO 122 J=1,2
          AJ(I1,J)=0.0
  122   CONTINUE
  123 CONTINUE
```

Solve deflections

```
C
      CALL SOLVE (NJOINT,IBAND)
```

Member and structure deflections and deflection influence parameters

```
C     MEMBER END ACTIONS
C
      DO 127 N=1,2
        DO 126 I=1,NMEM
          CALL INDEX (I)
          DO 124 IA=1,4
            M=IN(IA)
            D(IA)=DJ(M,N)
  124     CONTINUE
          CALL FORMSM (I)
          CALL FORMRT (I)
          CALL MULT (SM,4,4,RT,4,4,TEMP)
          CALL MULT (TEMP,4,4,D,4,1,AMT)
          DO 125 IA=1,4
            M=4*(I-1)+IA
            AM(M,N)=AM(M,N)+AMT(IA)
  125     CONTINUE
  126   CONTINUE
  127 CONTINUE
      DO 128 I=1,NMEM
        RNN(I)=0.0
        RN(I)=0.0
  128 CONTINUE
      DO 129 M=1,NMEM
        L=4*(M-1)+3
```

Deflection check

```
      RNN(M)=AM(L,1)
      RN(M)=AM(L,2)
  129 CONTINUE
C
C     MEMBER DEFLECTION CONTRIBUTION
C     DEFLECTION INFLUENCE PARAMETER
C     COMBINE MEMBER GROUPS
C
      CALL DEFL (DEFS,DEFPS,NMEM)
      CALL GROUP (NMEM,NUMID)
      WRITE (2,172)
      WRITE (2,173)
      DO 130 I=1,NMEM
         WRITE (2,174) I,IDENT(I),RNN(I),RN(I),ALEN(I),AA(I),DEF(I),DEFP
     1   (I),DEFPG(I)
  130 CONTINUE
      WRITE (2,175) DEFS
      WRITE (2,176) DEFPS
      IF (DEFS.LE.DMAXH) GO TO 147
      DO 132 I=1,NUMID
         DO 131 J=1,NMEM
            IF (IDENT(J).NE.I) GO TO 131
            DEFP(J)=DEFPG(J)
  131    CONTINUE
  132 CONTINUE
C
C     ORDERED DEFLECTION INFLUENCE PARAMETERS
C
      DO 133 I=1,NMEM
         H(I)=0.0
         T(I)=DEFP(I)
  133 CONTINUE
      DO 136 MM=1,NMEM
         DO 134 I=1,NMEM
            IF (T(I).GT.H(MM)) H(MM)=DEFP(I)
  134    CONTINUE
         DO 135 J=1,NMEM
            IF (T(J).LT.H(MM)) GO TO 135
            T(J)=-1.0
            GO TO 136
  135    CONTINUE
  136 CONTINUE
C
C     INCREASE SELECTED MEMBER AREAS
C
      DO 137 I=1,NMEM
         RA(I)=AA(I)
         T(I)=AA(I)
  137 CONTINUE
      NTR=1
  138 NTR=NTR+1
      MDEFP=H(NTR)
      IF (H(NTR).EQ.H(NTR-1)) GO TO 138
      DEFC=0.0
      DEFNC=0.0
      DO 140 I=1,NMEM
         IF (DEFP(I).LE.MDEFP) GO TO 140
         RA(I)=AA(I)*SQRT(DEFP(I)/MDEFP)
         DEFP(I)=MDEFP
         IF (IDENT(I).EQ.0) GO TO 140
         DO 139 J=1,NMEM
```

Revise member areas

```
            IF (IDENT(J).NE.IDENT(I)) GO TO 139
            IF (RA(J).LT.RA(I)) RA(J)=RA(I)
            DEFP(J)=DEFP(I)
  139    CONTINUE
  140 CONTINUE
      DO 142 I=1,NMEM
         IF (RA(I).EQ.AA(I)) GO TO 141
         DEFI=DEF(I)
         DEF(I)=DEFI*(AA(I)/RA(I))
         DEFC=DEFC+DEF(I)
         DEFS=(DEFS+DEF(I))-DEFI
         GO TO 142
  141    DEFNC=DEFNC+DEF(I)
  142 CONTINUE
      DO 143 I=1,NMEM
         AA(I)=RA(I)
  143 CONTINUE
      IF (DEFS.GT.DMAX) GO TO 138
      DEFN=DMAX-DEFNC
      DEFIP=MDEFP*((DEFN/DEFC)**2)
      DO 144 I=1,NMEM
         IF (T(I).EQ.AA(I)) GO TO 144
         AA(I)=AA(I)*SQRT(MDEFP/DEFIP)
  144 CONTINUE
      CALL DEFL (DEFS,DEFPS,NMEM)
      CALL GROUP (NMEM,NUMID)
      WRITE (2,177)
      WRITE (2,173)
      DO 145 I=1,NMEM
         WRITE (2,174) I,IDENT(I),RNN(I),RN(I),ALEN(I),AA(I),DEF(I),DEFP
     1   (I),DEFPG(I)
  145 CONTINUE
      WRITE (2,178) DEFS
      WRITE (2,179) DEFPS
      VOLUME=0.0
      DO 146 I=1,NMEM
         VOLUME=VOLUME+(AA(I)*ALEN(I))
  146 CONTINUE
      WRITE (2,159) VOLUME
```

Structure type

```
      IF (NTYPE.EQ.0) GO TO 147
      GO TO 111
  147 CONTINUE
      WRITE (2,180)
C
```

Format Statements

```
  148 FORMAT (8A10)
  149 FORMAT (1H1,///,21X,8A10)
  150 FORMAT (2I5)
  151 FORMAT (1H0,20X, 19HNUMBER OF JOINTS  =,I5)
  152 FORMAT (1H0,20X, 19HNUMBER OF MEMBERS =,I5)
  153 FORMAT (I5,2F10.2,2I5)
  154 FORMAT (1H0,//,21X,  5HJOINT,5X,  5HX(IN),5X,  5HY(IN),5X,  5HXTFI
     1X,5X,  5HYTFIX,/)
  155 FORMAT (21X,I5,2F10.2,2I10)
  156 FORMAT (3I5,2F10.2,I5)
  157 FORMAT (1H0,//,21X,  6HMEMBER,6X,  4HJEND,6X,  4HKEND,5X,  5HL(IN)
     1,4X,  6HA(IN2),4X,  6HE(KSI),5X,  5HIDENT,/)
  158 FORMAT (21X,I6,2I10,3F10.2,I10)
  159 FORMAT (1H0,20X, 23HSTRUCTURAL STEEL VOLUME,2X,F10.2,  6H (IN3))
  160 FORMAT (I5)
  161 FORMAT (1H0,//,21X, 19HAPPLIED JOINT LOADS)
  162 FORMAT (1H0,20X,  5HJOINT,5X, 10HX FORCE(K),5X, 10HY FORCE(K),/)
```

```
  163 FORMAT (I5,2F10.2)                                                  A 3060
  164 FORMAT (21X,I5,2F15.3)                                              A 3070
  165 FORMAT (1H0,//,21X, 17HAPPLIED UNIT LOAD)                           A 3080
  166 FORMAT (1H0,20X,  5HJOINT,8X,  7HX FORCE,8X,  7HY FORCE,/)          A 3090
  167 FORMAT (2F10.2,I5)                                                  A 3100
  168 FORMAT (1H0,//,21X, 28HMAXIMUM ALLOWABLE DEFLECTION,2X,F7.5,  5H (  A 3110
     1IN))                                                                A 3120
  169 FORMAT (1H0,20X, 28HDEFLECTION CONVERGENCE LIMIT,2X,F7.5,  5H (IN)  A 3130
     1)                                                                   A 3140
  170 FORMAT (1H0,20X, 33HINDETERMINATE STRUCTURAL ANALYSIS)              A 3150
  171 FORMAT (1H0,20X, 31HDETERMINATE STRUCTURAL ANALYSIS)                A 3160
  172 FORMAT (1H0,//,21X, 23HDEFLECTION CALCULATIONS)                     A 3170
  173 FORMAT (1H0,20X,  6HMEMBER,3X,  5HIDENT,2X, 12HREAL LOAD(K),3X,  9  A 3180
     1HUNIT LOAD,7X,  5HL(IN),5X,  6HA(IN2),8X,  7HDEF(IN),4X, 11HDEF IN  A 3190
     2F PAR,3X, 13HIDENT INF PAR)                                         A 3200
  174 FORMAT (21X,I6,I8,F14.3,F12.3,F12.2,F11.2,2F15.8,F16.8)             A 3210
  175 FORMAT (1H0,20X, 42HSTRUCTURE DEFLECTION AT POINT OF UNIT LOAD,3X,  A 3220
     1F7.5,  5H (IN))                                                     A 3230
  176 FORMAT (1H0,20X, 40HSTRUCTURE DEFLECTION INFLUENCE PARAMETER,2X,F1  A 3240
     10.8)                                                                A 3250
  177 FORMAT (1H0,//,21X, 31HREVISED DEFLECTION CALCULATIONS)             A 3260
  178 FORMAT (1H0,20X, 50HREVISED STRUCTURE DEFLECTION AT POINT OF UNIT   A 3270
     1LOAD,3X,F7.5,  5H (IN))                                             A 3280
  179 FORMAT (1H0,20X, 48HREVISED STRUCTURE DEFLECTION INFLUENCE PARAMET  A 3290
     1ER,2X,F10.8)                                                        A 3300
  180 FORMAT (1H0,20X, 30HDEFLECTION CRITERION SATISFIED)                 A 3310
C                                                                         A 3320
      END                                                                 A 3330
```

Subroutine DEFL

```
      SUBROUTINE DEFL (DEFS,DEFPS,NMEM)
      COMMON /INFORMB/ ALEN(100),E(100),AA(100),SM(4,4)
      COMMON /INFORME/ RNN(100),RN(100),DEF(100),DEFP(100),DEFPG(100)
      DEFS=0.0
      DEFPS=0.0
      DO 101 I=1,NMEM
         DEF(I)=0.0
         DEFP(I)=0.0
  101 CONTINUE
      DO 102 I=1,NMEM
         DEF(I)=((RNN(I)*RN(I))*ALEN(I))/(AA(I)*E(I))
         DEFP(I)=(RNN(I)*RN(I))/((AA(I)*AA(I))*E(I))
         DEFS=DEFS+DEF(I)
         DEFPS=DEFPS+DEFP(I)
  102 CONTINUE
      RETURN
C
      END
```

Subroutine FORMRT

```
      SUBROUTINE FORMRT (M)
      COMMON /INFORMA/ RT(4,4),SINT(100),COST(100),RTRANS(4,4)
      DO 101 I=1,4
      DO 101 J=1,4
         RT(I,J)=0.0
  101 CONTINUE
      CT=COST(M)
      ST=SINT(M)
      RT(1,1)=CT
      RT(2,2)=CT
      RT(3,3)=CT
      RT(4,4)=CT
      RT(1,2)=ST
      RT(3,4)=ST
      RT(2,1)=-ST
      RT(4,3)=-ST
C
C     ROTATION MATRIX TRANSPOSE
C
      DO 102 I=1,4
      DO 102 J=1,4
         RTRANS(J,I)=RT(I,J)
  102 CONTINUE
      RETURN
C
      END
```

Subroutine MULT

```
      SUBROUTINE MULT (A,MA,NA,B,MB,NB,C)
      DIMENSION A(4,4), B(4,4), C(4,4)
      DO 102 I=1,MA
      DO 102 J=1,NB
         SUM=0.0
         DO 101 L=1,NA
            SUM=SUM+A(I,L)*B(L,J)
  101    CONTINUE
         C(I,J)=SUM
  102 CONTINUE
      RETURN
C
      END
```

Subroutine FORMSM

```
      SUBROUTINE FORMSM (M)
      COMMON /INFORMB/ ALEN(100),E(100),AA(100),SM(4,4)
      AL=ALEN(M)
      A=(AA(M)*E(M))/AL
      DO 101 I=1,4
      DO 101 J=1,4
         SM(I,J)=0.0
  101 CONTINUE
      SM(1,1)=A
      SM(3,3)=A
      SM(1,3)=-A
      SM(3,1)=-A
      RETURN
C
      END
```

Subroutine INDEX

```
      SUBROUTINE INDEX (M)
      COMMON /INFORMC/ JC(50),KC(50),IN(4)
      JJ=JC(M)
      KK=KC(M)
      J2=2*JJ
      K2=2*KK
      IN(2)=J2
      IN(1)=J2-1
      IN(4)=K2
      IN(3)=K2-1
      RETURN
C
      END
```

Subroutine SOLVE

```
      SUBROUTINE SOLVE (NJOINT,IBAND)
      COMMON /INFORMD/ S(100,20),AJ(100,2),DJ(100,2)
C
C     GAUSSIAN ELIMINATION
C
      NEQ=2*NJOINT
      NEQ1=NEQ-1
      DO 105 I=1,NEQ1
         DIAG=S(I,1)
         JEND=NEQ-I+1
         IF (JEND.GT.IBAND) JEND=IBAND
         DO 103 J=2,JEND
            JROW=I+J-1
            FAC=S(I,J)/DIAG
            JCOL=0
            DO 101 K=J,JEND
               JCOL=JCOL+1
               S(JROW,JCOL)=S(JROW,JCOL)-S(I,K)*FAC
  101       CONTINUE
            DO 102 K=1,2
               AJ(JROW,K)=AJ(JROW,K)-AJ(I,K)*FAC
  102       CONTINUE
            S(I,J)=FAC
  103    CONTINUE
         DO 104 K=1,2
            AJ(I,K)=AJ(I,K)/DIAG
  104    CONTINUE
  105 CONTINUE
      DO 106 K=1,2
         AJ(NEQ,K)=AJ(NEQ,K)/S(NEQ,1)
  106 CONTINUE
C
C     BACK SUBSTITUTION
C
      DO 109 K=1,2
         DJ(NEQ,K)=AJ(NEQ,K)
         DO 108 I=1,NEQ1
            JROW=NEQ-I
            JEND=NEQ-JROW+1
            IF (JEND.GT.IBAND) JEND=IBAND
            RHS=AJ(JROW,K)
            DO 107 J=2,JEND
               JCOL=JROW+J-1
               RHS=RHS-S(JROW,J)*DJ(JCOL,K)
  107       CONTINUE
            DJ(JROW,K)=RHS
  108    CONTINUE
  109 CONTINUE
      RETURN
C
      END
```

Subroutine GROUP

```
      SUBROUTINE GROUP (NMEM,NUMID)
      COMMON /INFORMB/ ALEN(100),E(100),AA(100),SM(4,4)
      COMMON /INFORME/ RNN(100),RN(100),DEF(100),DEFP(100),DEFPG(100)
      COMMON /INFORMF/ IDENT(100),TVOL(100),TDEF(100),AVDEFP(100)
      DO 101 I=1,NMEM
         DEFPG(I)=0.0
  101 CONTINUE
      DO 104 I=1,NUMID
         TVOL(I)=0.0
         TDEF(I)=0.0
         AVDEFP(I)=0.0
         DO 102 J=1,NMEM
            IF (IDENT(J).NE.I) GO TO 102
            TVOL(I)=TVOL(I)+(AA(J)*ALEN(J))
            TDEF(I)=TDEF(I)+DEF(J)
  102    CONTINUE
         AVDEFP(I)=TDEF(I)/TVOL(I)
         DO 103 J=1,NMEM
            IF (IDENT(J).NE.I) GO TO 103
            DEFPG(J)=AVDEFP(I)
  103    CONTINUE
  104 CONTINUE
      RETURN
C
      END
```

B.15 Storage Location Revisions

Member Modifications

Single storage arrays dependent on the number of members (M), currently 100, are:

SINT(100)	RNN(100)
COST(100)	RN(100)
ALEN(100)	DEF(100)
E(100)	DEFP(100)
AA(100)	DEFPG(100)
RA(100)	H(100)
IDENT(100)	T(100)

Note that no program modifications are required if the number of members remains less than or equal to 100.

The only multidimensional storage array dependent on the number of members is the member end actions matrix, AM(400,2). The 400 represents 100 members times 4 member end actions, while the 2 specifies the number of loading cases (real and unit loading systems respectively).

Joint Modifications

Single storage arrays dependent on the number of joints (JN), currently 50, are:

JC(50)	Y(50)
KC(50)	XTFIX(50)
X(50)	YTFIX(50)

Multidimensional storage arrays dependent on the number of joints are:

S(100,20)

AJ(100,2)

PA(100,2)

DJ(100,2)

In each of these four matrices, the 100 represents 50 joints times 2 independent translations per joint. The second parameter in the S matrix specifies the maximum IBAND limit (see Appendix B.6). The second parameter in the AJ, PA, and DJ matrices specifies the total number of loading cases to be analyzed.

Stiffness Matrice Modifications

As long as the program is to remain a plane truss analysis-design routine, member stiffness matrice modifications need not be attempted. The current method of solution employs the following storage arrays to allow axial loads and shear at each member end:

IN(4)	RT(4,4)
JTL(4)	RTRANS(4,4)
D(4)	SM(4,4)
AMT(4)	TEMP(4,4)
	SMR(4,4)

This concludes the listing of storage arrays necessary to insure program operation in its current form. When revising any array dimensions, all COMMON and DIMENSION statements throughout the main program and its subprograms must be changed to reflect the desired modifications.

APPENDIX C

DESIGN EXAMPLES

C.1 Design Example 1

Design Example 1 involves the design, analysis, and optimization of the one-bay three-story truss system shown in Fig. C.1a. This braced frame was selected primarily to illustrate gravity and lateral load design procedures, in addition to demonstrating the use of Program WTD1 for drift computation and member area modification. This particular truss is assigned a rather restrictive structure deflection index of H/800 to necessitate the use of the program's design and optimization routines. The truss as designed for strength considerations has a computed deflection index of H/735.

In this and all other design examples, the most recent AISC specifications are used. Assumptions employed in the various designs are specified as needed. For this example, a listing of the data file required for program execution is shown following the design calculations and preceeding the program output.

STRUCTURAL CONFIGURATION

The elevation and plan views for Design Example 1 are shown in Fig. 1.Ca and b respectively. The building is rectangular in plan, 75 ft by 56 ft, with two wind trusses per direction acting to resist the applied lateral forces. Therefore, each truss in the direction of the applied lateral load (Fig. C.1b) has an effective wind width of 37.5 ft. Structural members in this and all other design frames are assumed to be A36 steel.

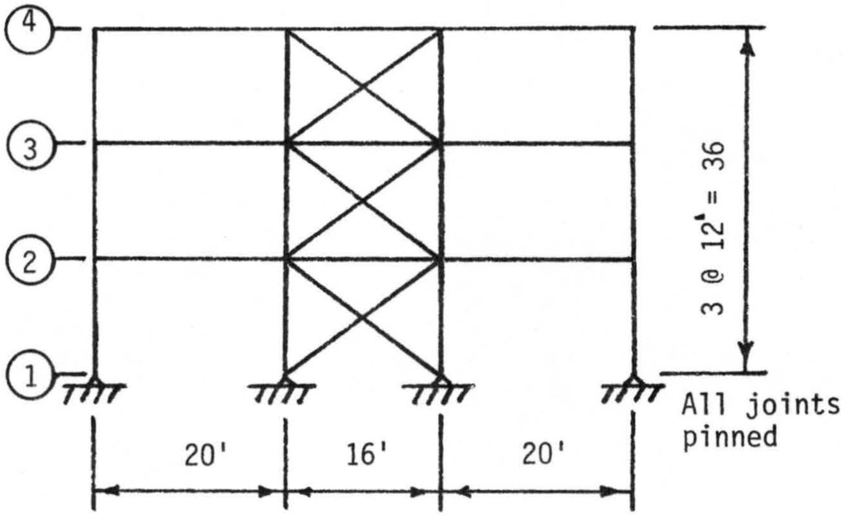

Fig. C.1a
Braced frame system
Design Example 1.

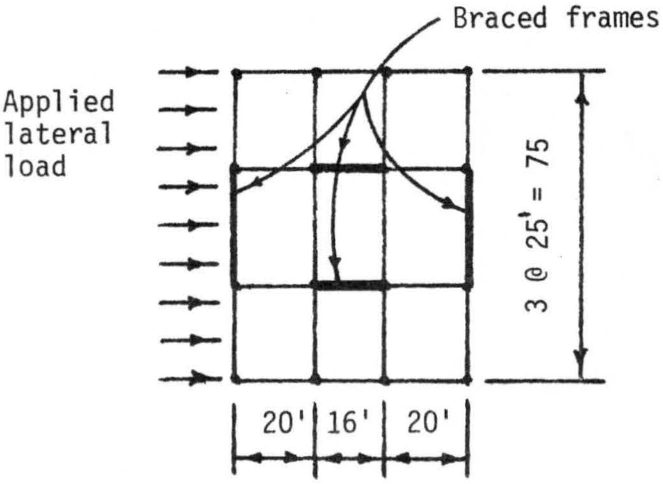

Fig. C.1b
Structure plan view
Design Example 1.

GRAVITY LOAD DESIGN

Loads

Live loads:	80 psf floors
	40 psf roof
	20 psf snow
Dead load:	50 psf all levels

Beam Design

Assume that beams and girders located perpendicular to the braced framing systems span 25 ft.

Beam	Beam load (k/ft)	Beam moment (k-ft)
Roof	2.75	$88 = w(1)^2/8$
Floors	3.25	104

Beam	Trial beam	M_R (k-ft)	Area (in^2)
Roof	W 16 X 31	94	9.13
Floors	W 18 X 35	116	10.30

Column Design

Tributary area for each floor level: 450 ft^2

All $KL_x = KL_y = 12$ft.

Level	Column load (k)	Trial column	P_{ALLOW} (k)	Area (in^2)
3-4	49.5	W 8 X 17	53	5.01
2-3	108.0	W 8 X 28	118	8.23
1-2	166.5	W 8 X 35	168	10.30

WIND LOAD DESIGN

Assume that a constant wind pressure of 20 psf is applied to the exposed face of the structure. Since each braced frame in the direction of concern must provide 50% of the structure's lateral support, the tributary wind width is 37.5 ft.

For simplified design, assume a counter bracing system, in which only the tension member is assumed to act in resisting wind forces (the compression diagonal with a large l/r ratio is assumed to buckle, thus providing no resistance).

Level	Wind force (k)	3/4 W (k)**
4	4.5	3.375
3	9.0	6.750
2	9.0	6.750

** see AISC Sec. 1.5.6

Member design loads resulting form the applied wind forces are summarized in Fig. C.2.

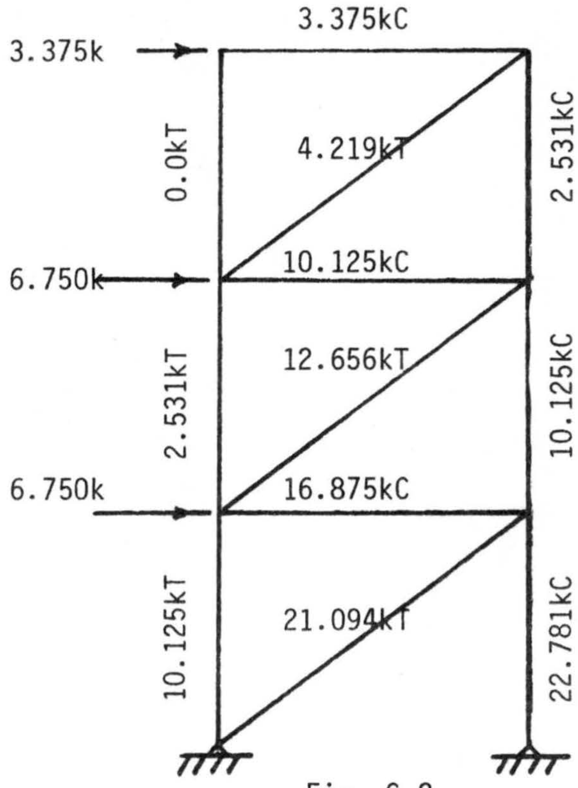

Fig. C.2

Member loads due to wind

Design Example 1.

COMBINED LOAD CASE DESIGN

Column Check

Level	3/4(W) k	3/4(D+L) k	3/4(D+L+W) k	(D+L)O.K.?
3-4	2.53	37.13	39.66	Yes
2-3	10.13	81.00	91.13	Yes
1-2	22.78	124.88	147.66	Yes

Beam Check

Assume floor slabs provide full lateral beam support.

AISC Eq. 1.6-1b: $\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} \leq 1.0$

Beam	3/4 Beam moment (k-ft)	AISC 1.6-1b
Roof	66	0.72 O.K.
Floor (level 3)	78	0.72 O.K.
Floor (level 2)	78	0.75 O.K.

Bracing Design

Assume $F_t = 0.6F_y$, compute member areas required to satisfy 3/4(W) forces.

Level	3/4(W) k	Area (in^2)
3-4	4.22	0.20
2-3	12.66	0.59
1-2	21.09	0.98

Assume minimum size bracing member (for fabrication purposes) is L 3 x 2 x 3/16.

Level	Bracing member	Area (in^2)
3-4	L 3 x 2 x 3/16	0.902
2-3	L 3 x 2 x 3/16	0.902
1-2	L 3 x 2 x 1/4	1.19

STRUCTURE DRIFT LIMIT

Maximum allowable lateral deflection at top of structure due to full wind load: H/800 = 0.540 in.

PROGRAM WTD1 DATA FILE MODEL

Fig. C.3 defines the member and joint numbering systems incorporated in the structure's data file. Note that the maximum JEND-KEND difference is 3. For this example, the joint numbering shown results in the least possible stiffness matrix bandwidth.

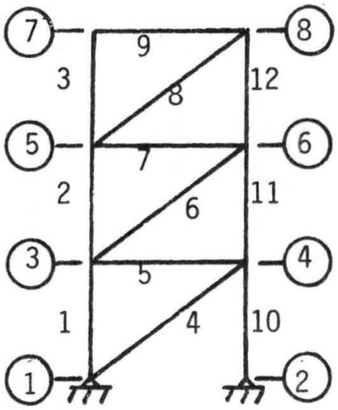

Fig. C.3
Structure model
Design Example 1.

Program WTD1 Data File - Example 1

```
DESIGN EXAMPLE 1 -- 1 BAY 3 STORY DETERMINATE TRUSS
    8   12
    1          0.0       0.0    1    1
    2        192.0       0.0    1    1
    3          0.0     144.0    0    0
    4        192.0     144.0    0    0
    5          0.0     288.0    0    0
    6        192.0     288.0    0    0
    7          0.0     432.0    0    0
    8        192.0     432.0    0    0
    1    1    3      10.30      29000.0    1
    2    3    5       8.23      29000.0    2
    3    5    7       5.01      29000.0    3
    4    1    4       1.19      29000.0    0
    5    3    4      10.30      29000.0    0
    6    3    6      0.902      29000.0    0
    7    5    6      10.30      29000.0    0
    8    5    8      0.902      29000.0    0
    9    7    8       9.13      29000.0    0
   10    2    4      10.30      29000.0    1
   11    4    6       8.23      29000.0    2
   12    6    8       5.01      29000.0    3
    3           .0
    3          9.0
    5          9.0
    7          4.5
    7          1.0
     0.540
```

DESIGN EXAMPLE 1 -- 1 BAY 3 STORY DETERMINATE TRUSS

NUMBER OF JOINTS = 8

NUMBER OF MEMBERS = 12

JOINT	X(IN)	Y(IN)	XTFIX	YTFIX
1	0.00	0.00	1	1
2	192.00	0.00	1	1
3	0.00	144.00	0	0
4	192.00	144.00	0	0
5	0.00	288.00	0	0
6	192.00	288.00	0	0
7	0.00	432.00	0	0
8	192.00	432.00	0	0

MEMBER	JEND	KEND	L(IN)	A(IN2)	E(KSI)	IDENT
1	1	3	144.00	10.30	29000.00	1
2	3	5	144.00	8.23	29000.00	2
3	5	7	144.00	5.01	29000.00	3
4	1	4	240.00	1.19	29000.00	0
5	3	4	192.00	10.30	29000.00	0
6	3	6	240.00	.90	29000.00	0
7	5	6	192.00	10.30	29000.00	0
8	5	8	240.00	.90	29000.00	0
9	7	8	192.00	9.13	29000.00	0
10	2	4	144.00	10.30	29000.00	1
11	4	6	144.00	8.23	29000.00	2
12	6	8	144.00	5.01	29000.00	3

STRUCTURAL STEEL VOLUME 13206.24 (IN3)

APPLIED JOINT LOADS

JOINT	X FORCE(K)	Y FORCE(K)
3	9.000	0.000
5	9.000	0.000
7	4.500	0.000

APPLIED UNIT LOAD

JOINT	X FORCE	Y FORCE
7	1.000	0.000

MAXIMUM ALLOWABLE DEFLECTION .54000 (IN)

DEFLECTION CONVERGENCE LIMIT 0.00000 (IN)

DETERMINATE STRUCTURAL ANALYSIS

DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	13.500	1.500	144.00	10.30	.00976230	.00000658	.00001440
2	2	3.375	.750	144.00	8.23	.00152721	.00000129	.00000580
3	3	0.000	.000	144.00	5.01	0.00000000	0.00000000	.00000174
4	0	28.125	1.250	240.00	1.19	.24449435	.00085607	0.00000000
5	0	-22.500	-1.000	192.00	10.30	.01446267	.00000731	0.00000000
6	0	16.875	1.250	240.00	.90	.19353544	.00089401	0.00000000
7	0	-13.500	-1.000	192.00	10.30	.00867760	.00000439	0.00000000
8	0	5.625	1.250	240.00	.90	.06451181	.00029800	0.00000000
9	0	-4.500	-1.000	192.00	9.13	.00326321	.00000186	0.00000000
10	1	-30.375	-2.250	144.00	10.30	.03294777	.00002221	.00001440
11	2	-13.500	-1.500	144.00	8.23	.01221771	.00001031	.00000580
12	3	-3.375	-.750	144.00	5.01	.00250878	.00000348	.00000174

STRUCTURE DEFLECTION AT POINT OF UNIT LOAD .58791 (IN)

STRUCTURE DEFLECTION INFLUENCE PARAMETER .00210552

REVISED DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	13.500	1.500	144.00	10.30	.00976230	.00000658	.00001440
2	2	3.375	.750	144.00	8.23	.00152721	.00000129	.00000580
3	3	0.000	.000	144.00	5.01	0.00000000	0.00000000	.00000174
4	0	28.125	1.250	240.00	1.32	.21983639	.00069211	0.00000000
5	0	-22.500	-1.000	192.00	10.30	.01446267	.00000731	0.00000000
6	0	16.875	1.250	240.00	1.03	.17028454	.00069211	0.00000000
7	0	-13.500	-1.000	192.00	10.30	.00867760	.00000439	0.00000000
8	0	5.625	1.250	240.00	.90	.06451161	.00029800	0.00000000
9	0	-4.500	-1.000	192.00	9.13	.00326321	.00000186	0.00000000
10	1	-30.375	-2.250	144.00	10.30	.03294777	.00002221	.00001440
11	2	-13.500	-1.500	144.00	8.23	.01221771	.00001031	.00000580
12	3	-3.375	-.750	144.00	5.01	.00250878	.00000348	.00000174

REVISED STRUCTURE DEFLECTION AT POINT OF UNIT LOAD .54000 (IN)

REVISED STRUCTURE DEFLECTION INFLUENCE PARAMETER .00173965

STRUCTURAL STEEL VOLUME 13267.83 (IN3)

DEFLECTION CRITERION SATISFIED

PROGRAM WTD1 RESULTS

	Initial	Revised
Structure steel volume (in^3)	13206.2	13267.8
Structure deflection (in)	0.58791	0.54000
Structure defl. infl. parameter	0.00211	0.00174

Member Modifications

Member	Type	Area (in^2) Orig.	Rev.	New Section	Area(in^2)
4	Bracing	1.19	1.32	L 3(1/2) x 2(1/2) x 1/4	1.44
6	Bracing	0.90	1.03	L 3 x 2 x 1/4	1.19

Original braced frame volume with all members: 13923.8 in^3

Final braced frame steel volume (including opposite counter bracing members): 14047.8 in^3

Deflection decrease = 8.15%.

Volume increase = 0.89%.

Structural stiffness increase = 8.87%.

Note the sizeable structure deflection decrease (8.15%) provided by the smaller steel volume increase (0.89%). The final braced frame results for Design Example 1 are summarized in Fig. C.4.

The above method of manually accounting for the volume of the inactive diagonal member results in a near but not necessarily exact optimal solution. As shown in Sec. 4.4, inclusion of the volume of the inactive diagonal in the optimization program will result in relatively less volume increase assigned to the bracing when members other than the bracing are also modified. In this particular design example, only bracing member areas are increased, and the least weight solution has been reached. The zero force members can be considered in Program WTD1 by declaring them in an IDENT group with the other diagonal and assigning the zero force members a low modulus of elasticity so the analysis will show essentially no load in these members. Alternatively, the program could be modified to consider zero force members for group DEFP computation while omitting them from the analysis.

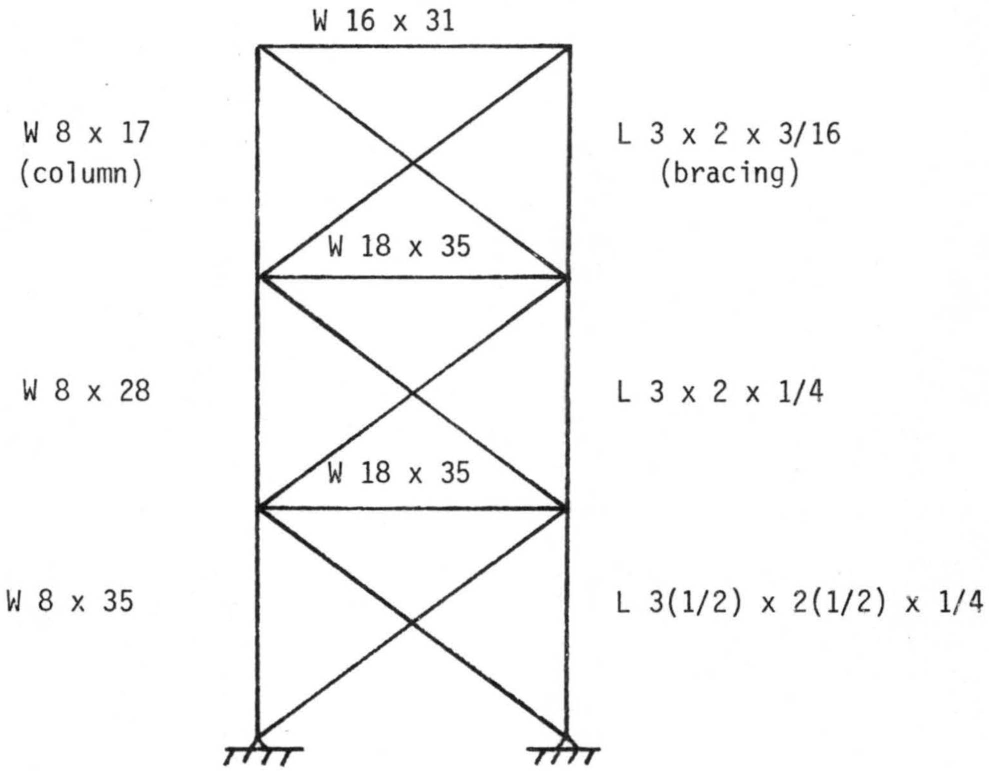

Fig. C.4
Final braced frame members
Design Example 1.

C.2 Design Example 2

Design Example 2 involves the design, analysis, and optimization of the one bay eight story truss system shown in Fig. C.5. This braced frame was selected to demonstrate more completely the program's design capabilities, including the member IDENT feature.

For design purposes, two story high columns are used in addition to a fully effective truss system in which both the bracing members in tension and compression are assumed to act in resisting lateral forces. This particular structure is assigned a maximum deflection index of H/600.

STRUCTURAL CONFIGURATION

The elevation view for Design Example 2 is shown in Fig. C.5. This building is rectangular in plan, 75 ft by 56 ft, with two wind trusses per direction acting to resist the applied lateral forces. As in Design Example 1, each truss in the direction of the applied lateral load has an effective wind width of 37.5 ft.

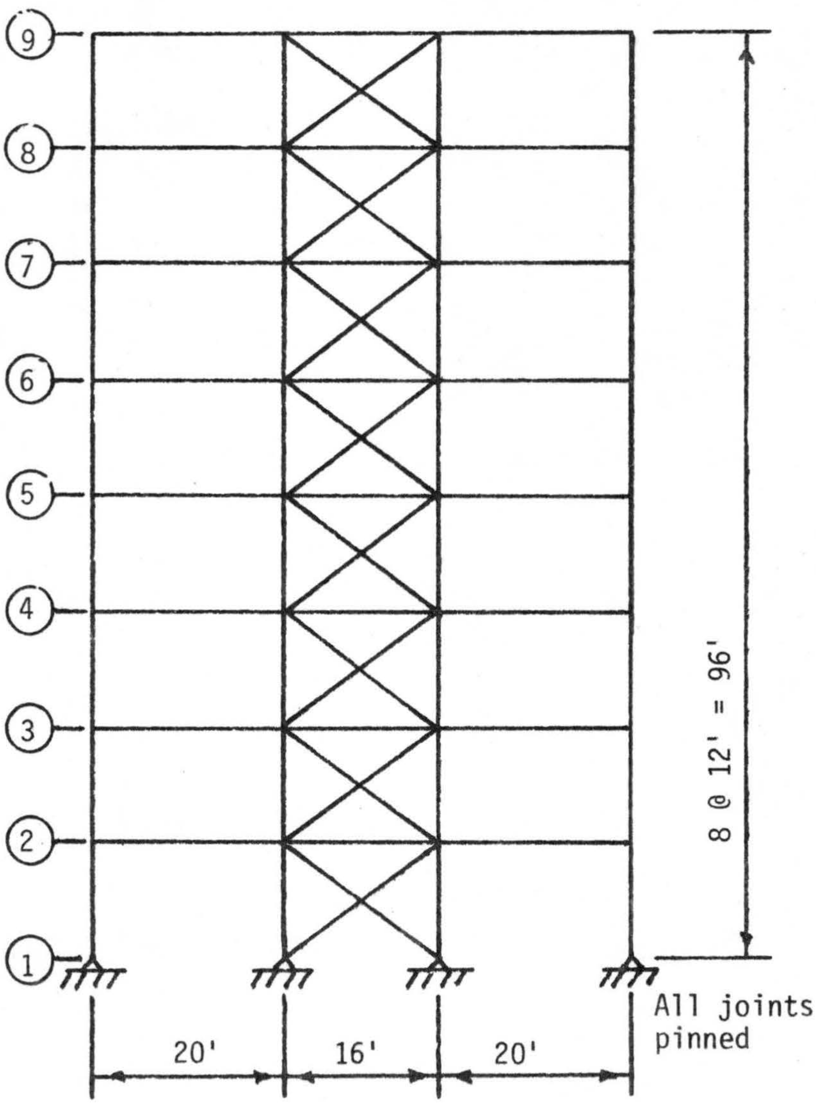

Fig. C.5
Braced frame system
Design Example 2.

GRAVITY LOAD DESIGN

Loads

Live load: 80 psf all levels, including roof

Dead load: 50 psf all levels

Beam Design

Assume, as in Design Example 1, that beams and girders located perpendicular to the braced frames span 25 ft. Therefore, the maximum beam moment is 104 k-ft and the most economical section proves to be a W 18 x 35.

Column Design

Tributary area for each two story column stack: 900 ft^2

Level	Column load (k)	Trial column	P_{ALLOW} (k)
7-9	117	W 8 x 28	118
5-7	234	W 10 x 49	255
3-5	351	W 12 x 65	354
1-3	468	W 14 x 87	493

WIND LOAD DESIGN

Assume the wind pressure distribution shown in Fig. C.6 is applied to the exposed face of the structure. Using a tributary wind width of 37.5 ft (see Design Example 1), the structure design wind forces are as summarized in Fig. C.7.

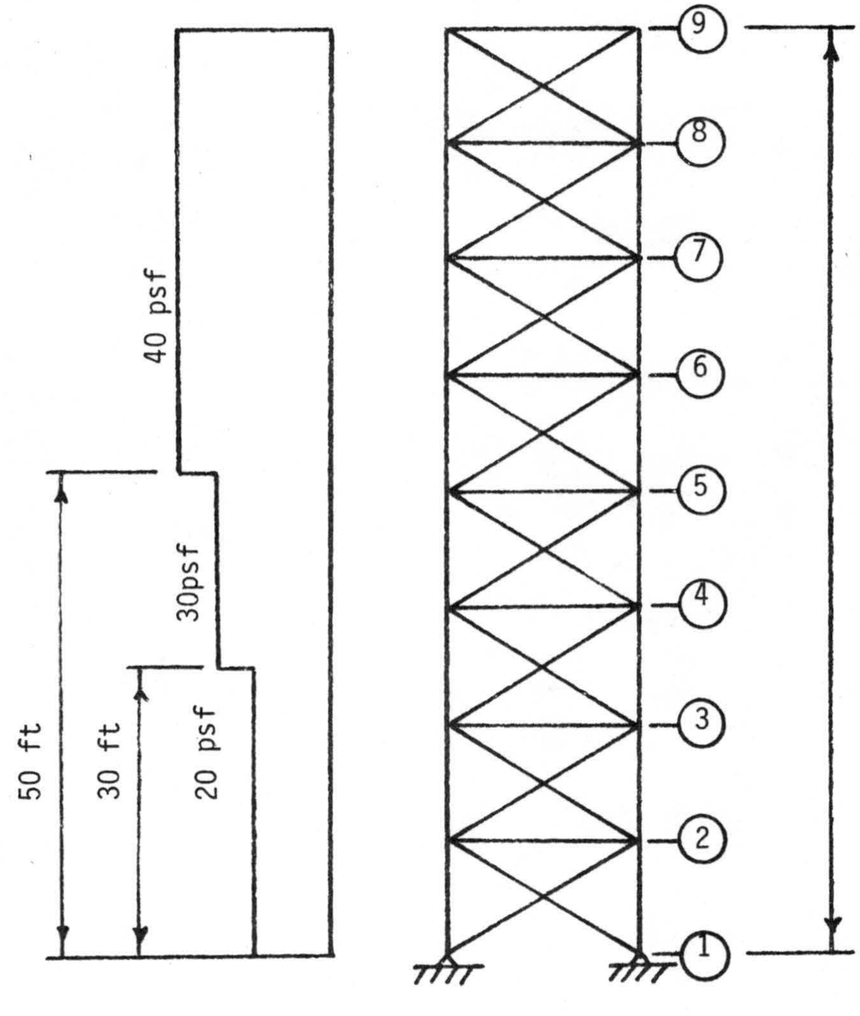

Fig. C.6
Wind pressure distribution
, Design Example 2.

Level	Wind force (k)	3/4 W (k)
9	9.0	6.75
8	18.0	13.50
7	18.0	13.50
6	18.0	13.50
5	15.0	11.25
4	13.5	10.13
3	9.0	6.75
2	9.0	6.75

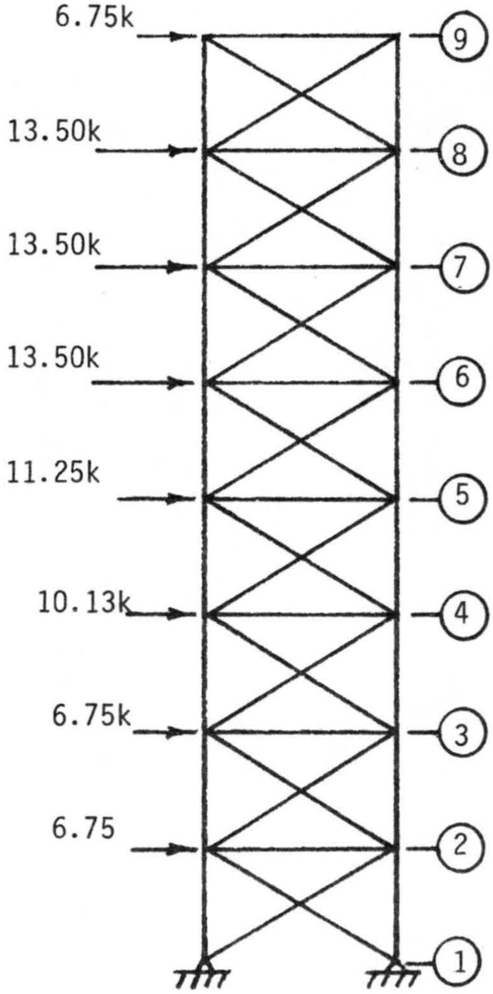

Fig. C.7

Design wind forces

Design Example 2.

Bracing Design

The fully effective truss employed in this structure, was designed assuming that each bracing member on each floor level of the indeterminate truss accepts 50% of the cumulative lateral shear. The design procedure for each level involved computing the design force in the compression diagonal, selecting a trial section, and calculating the maximum allowable load based on the appropriate slenderness ratio.

Level	3/4 W (k)	Trial member	P_{ALLOW} (k)
8-9	4.22	*L 3(1/2) x 3 x 1/4	9.92
7-8	12.66	L 4 x 3 x 5/16	13.48
6-7	21.09	L 4 x 3(1/2) x 7/16	21.78
5-6	29.53	L 4 x 3(1/2) x 5/8	30.10
4-5	36.56	L 6 x 4 x 1/2	40.95
3-4	42.89	L 6 x 4 x 9/16	45.52
2-3	47.11	L 6 x 4 x 5/8	50.00
1-2	51.33	L 7 x 4 x 5/8	55.49

* indicates member size controlled by AISC 1.8.4.

COMBINED LOAD CASE DESIGN

Column Check

Level	3/4(W) k	3/4(D+L) k	3/4(D+L+W) k	(D+L)O.K.?
7-9	12.65	87.75	100.40	Yes
5-7	63.28	175.50	238.78	Yes
3-5	150.61	263.25	413.86	No (354)
1-3	263.67	351.00	614.67	No (493)

Level	New column	P_{ALLOW} (k)	Area (in^2)
3-5	W 12 x 79	431	23.2
1-3	W 14 x 111	631	32.7

Beam Check

Level	3/4 W (k)	AISC Eq. 1.6-1b
9	3.37	0.69 O.K.
8	10.13	0.72 O.K.
7	16.88	0.75 O.K.
6	23.63	0.78 O.K.
5	29.25	0.80 O.K.
4	34.31	0.83 O.K.
3	37.69	0.84 O.K.
2	41.06	0.86 O.K.

STRUCTURE DRIFT LIMIT

Maximum allowable lateral deflection at top of structure due to full wind load: H/600 = 1.920 in.

PROGRAM WTD1 DATA FILE MODEL

Fig. C.8 defines the member and joint numbering systems incorporated in the structure's data file. Note that the maximum JEND-KEND difference is 3. For this example, the joint numbering shown results in the least possible stiffness matrix bandwidth.

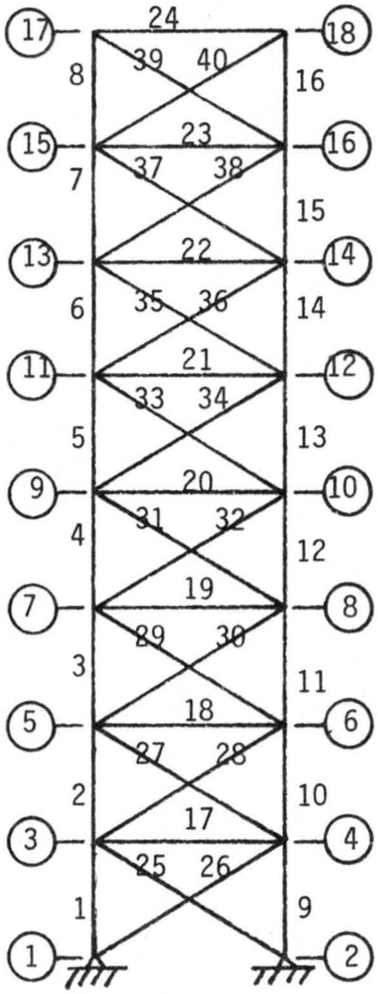

Fig. C.8
Structure model
Design Example 2.

DESIGN EXAMPLE 2 -- 1 BAY 8 STORY INDETERMINATE TRUSS

NUMBER OF JOINTS = 18

NUMBER OF MEMBERS = 40

JOINT	X(IN)	Y(IN)	XTFIX	YTFIX
1	0.00	0.00	1	1
2	192.00	0.00	1	1
3	0.00	144.00	0	0
4	192.00	144.00	0	0
5	0.00	288.00	0	0
6	192.00	288.00	0	0
7	0.00	432.00	0	0
8	192.00	432.00	0	0
9	0.00	576.00	0	0
10	192.00	576.00	0	0
11	0.00	720.00	0	0
12	192.00	720.00	0	0
13	0.00	864.00	0	0
14	192.00	864.00	0	0
15	0.00	1008.00	0	0
16	192.00	1008.00	0	0
17	0.00	1152.00	0	0
18	192.00	1152.00	0	0

MEMBER	JEND	KEND	L(IN)	A(IN2)	E(KSI)	IDENT
1	1	3	144.00	32.70	29000.00	1
2	3	5	144.00	32.70	29000.00	1
3	5	7	144.00	23.20	29000.00	2
4	7	9	144.00	23.20	29000.00	2
5	9	11	144.00	14.40	29000.00	3
6	11	13	144.00	14.40	29000.00	3
7	13	15	144.00	8.23	29000.00	4
8	15	17	144.00	8.23	29000.00	4
9	2	4	144.00	32.70	29000.00	1
10	4	6	144.00	32.70	29000.00	1
11	6	8	144.00	23.20	29000.00	2
12	8	10	144.00	23.20	29000.00	2
13	10	12	144.00	14.40	29000.00	3
14	12	14	144.00	14.40	29000.00	3
15	14	16	144.00	8.23	29000.00	4
16	16	18	144.00	8.23	29000.00	4
17	3	4	192.00	10.30	29000.00	0
18	5	6	192.00	10.30	29000.00	0
19	7	8	192.00	10.30	29000.00	0

Program WTD1 Output - Example 2

20	9	10	192.00	10.30	29000.00	0
21	11	12	192.00	10.30	29000.00	0
22	13	14	192.00	10.30	29000.00	0
23	15	16	192.00	10.30	29000.00	0
24	17	18	192.00	10.30	29000.00	0
25	3	2	240.00	6.48	29000.00	5
26	1	4	240.00	6.48	29000.00	5
27	5	4	240.00	5.86	29000.00	6
28	3	6	240.00	5.86	29000.00	6
29	7	6	240.00	5.31	29000.00	7
30	5	8	240.00	5.31	29000.00	7
31	9	8	240.00	4.75	29000.00	8
32	7	10	240.00	4.75	29000.00	8
33	11	10	240.00	4.30	29000.00	9
34	9	12	240.00	4.30	29000.00	9
35	13	12	240.00	3.09	29000.00	10
36	11	14	240.00	3.09	29000.00	10
37	15	14	240.00	2.09	29000.00	11
38	13	16	240.00	2.09	29000.00	11
39	17	16	240.00	1.56	29000.00	12
40	15	18	240.00	1.56	29000.00	12

STRUCTURAL STEEL VOLUME 77105.28 (IN3)

APPLIED JOINT LOADS

JOINT	X FORCE(K)	Y FORCE(K)
3	9.000	0.000
5	9.000	0.000
7	13.500	0.000
9	15.000	0.000
11	18.000	0.000
13	18.000	0.000
15	18.000	0.000
17	9.000	0.000

APPLIED UNIT LOAD

JOINT	X FORCE	Y FORCE
17	1.000	0.000

MAXIMUM ALLOWABLE DEFLECTION 1.92000 (IN)

DEFLECTION CONVERGENCE LIMIT 0.00000 (IN)

INDETERMINATE STRUCTURAL ANALYSIS

DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	351.931	5.625	144.00	32.70	.30060524	.00006384	.00005333
2	1	273.435	4.875	144.00	32.70	.20241597	.00004299	.00005333
3	2	201.514	4.125	144.00	23.20	.17791187	.00005325	.00004137
4	2	138.088	3.375	144.00	23.20	.09974835	.00002986	.00004137
5	3	85.275	2.625	144.00	14.40	.07718864	.00003722	.00002526
6	3	44.646	1.875	144.00	14.40	.02886643	.00001392	.00002526
7	4	17.431	1.124	144.00	8.23	.01182518	.00000998	.00000516
8	4	3.707	.388	144.00	8.23	.00086778	.00000073	.00000516
9	1	-351.194	-5.625	144.00	32.70	.29997607	.00006371	.00005333
10	1	-272.190	-4.875	144.00	32.70	.20149497	.00004279	.00005333
11	2	-200.111	-4.125	144.00	23.20	.17667382	.00005288	.00004137
12	2	-136.412	-3.375	144.00	23.20	.09853792	.00002950	.00004137
13	3	-83.475	-2.625	144.00	14.40	.07555921	.00003644	.00002526
14	3	-43.104	-1.875	144.00	14.40	.02786850	.00001344	.00002526
15	4	-16.319	-1.126	144.00	8.23	.01108263	.00000935	.00000516
16	4	-3.043	-.362	144.00	8.23	.00065465	.00000056	.00000516
17	0	-3.180	.000	192.00	10.30	-.00000000	-.00000000	0.00000000
18	0	-2.736	-.000	192.00	10.30	.00000000	.00000000	0.00000000
19	0	-4.698	.000	192.00	10.30	-.00000000	-.00000000	0.00000000
20	0	-5.183	-.000	192.00	10.30	.00000001	.00000000	0.00000000
21	0	-6.772	.000	192.00	10.30	-.00000020	-.00000000	0.00000000
22	0	-7.231	-.001	192.00	10.30	.00000339	.00000000	0.00000000
23	0	-7.817	.017	192.00	10.30	-.00008330	-.00000004	0.00000000
24	0	-4.058	-.483	192.00	10.30	.00125884	.00000064	0.00000000
25	5	-69.051	-.625	240.00	6.48	.05511760	.00003544	.00003513
26	5	67.824	.625	240.00	6.48	.05413767	.00003481	.00003513
27	6	-63.849	-.625	240.00	5.86	.05635749	.00004007	.00003942
28	6	61.776	.625	240.00	5.86	.05452723	.00003877	.00003942
29	7	-58.356	-.625	240.00	5.31	.05684402	.00004460	.00004371
30	7	56.019	.625	240.00	5.31	.05456743	.00004282	.00004371
31	8	-50.146	-.625	240.00	4.75	.05460589	.00004790	.00004657
32	8	47.354	.625	240.00	4.75	.05156471	.00004523	.00004657
33	9	-40.875	-.625	240.00	4.30	.04916792	.00004764	.00004590
34	9	37.875	.625	240.00	4.30	.04555939	.00004415	.00004590
35	10	-29.410	-.625	240.00	3.09	.04923453	.00006639	.00006348
36	10	26.840	.625	240.00	3.09	.04492392	.00006058	.00006348
37	11	-17.801	-.624	240.00	2.09	.04398653	.00008769	.00008325
38	11	15.949	.626	240.00	2.09	.03953213	.00007881	.00008325
39	12	-6.178	-.647	240.00	1.56	.02119483	.00005661	.00004998
40	12	5.072	.603	240.00	1.56	.01623352	.00004336	.00004998

STRUCTURE DEFLECTION AT POINT OF UNIT LOAD 2.54002 (IN)

STRUCTURE DEFLECTION INFLUENCE PARAMETER .00131593

REVISED DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	351.931	5.625	144.00	47.09	.20874154	.00003078	.00002572
2	1	273.435	4.875	144.00	47.09	.14055850	.00002073	.00002572
3	2	201.514	4.125	144.00	29.43	.14026470	.00003310	.00002572
4	2	138.088	3.375	144.00	29.43	.07864103	.00001856	.00002572
5	3	85.275	2.625	144.00	14.40	.07718864	.00003722	.00002526
6	3	44.646	1.875	144.00	14.40	.02886643	.00001392	.00002526
7	4	17.431	1.124	144.00	8.23	.01182518	.00000998	.00000516
8	4	3.707	.388	144.00	8.23	.00066778	.00000073	.00000516
9	1	-351.194	-5.625	144.00	47.09	.20830464	.00003072	.00002572
10	1	-272.190	-4.875	144.00	47.09	.13991896	.00002063	.00002572
11	2	-200.111	-4.125	144.00	29.43	.13928863	.00003287	.00002572
12	2	-136.412	-3.375	144.00	29.43	.07768673	.00001833	.00002572
13	3	-83.475	-2.625	144.00	14.40	.07555921	.00003644	.00002526
14	3	-43.104	-1.875	144.00	14.40	.02786850	.00001344	.00002526
15	4	-16.319	-1.126	144.00	8.23	.01108263	.00000935	.00000516
16	4	-3.043	-.362	144.00	8.23	.00066465	.00000056	.00000516
17	0	-3.180	.000	192.00	10.30	-.00000000	-.00000000	0.00000000
18	0	-2.736	-.000	192.00	10.30	.00000000	.00000000	0.00000000
19	0	-4.698	.000	192.00	10.30	-.00000000	-.00000000	0.00000000
20	0	-5.183	-.000	192.00	10.30	.00000001	.00000000	0.00000000
21	0	-6.772	.000	192.00	10.30	-.00000020	-.00000000	0.00000000
22	0	-7.231	-.001	192.00	10.30	.00000339	.00000000	0.00000000
23	0	-7.817	.017	192.00	10.30	-.00008330	-.00000004	0.00000000
24	0	-4.058	-.483	192.00	10.30	.00125884	.00000064	0.00000000
25	5	-69.051	-.625	240.00	7.57	.04716043	.00002595	.00002572
26	5	67.824	.625	240.00	7.57	.04632197	.00002549	.00002572
27	6	-63.849	-.625	240.00	7.26	.04551827	.00002614	.00002572
28	6	61.776	.625	240.00	7.26	.04404002	.00002529	.00002572
29	7	-58.356	-.625	240.00	6.92	.04360018	.00002624	.00002572
30	7	56.019	.625	240.00	6.92	.04185400	.00002519	.00002572
31	8	-50.146	-.625	240.00	6.39	.04057937	.00002645	.00002572
32	8	47.354	.625	240.00	6.39	.03831937	.00002498	.00002572
33	9	-40.875	-.625	240.00	5.74	.03680439	.00002670	.00002572
34	9	37.875	.625	240.00	5.74	.03410324	.00002474	.00002572
35	10	-29.410	-.625	240.00	4.85	.03133578	.00002689	.00002572
36	10	26.840	.625	240.00	4.85	.02859225	.00002454	.00002572
37	11	-17.801	-.624	240.00	3.76	.02444681	.00002709	.00002572
38	11	15.949	.626	240.00	3.76	.02197115	.00002434	.00002572
39	12	-6.178	-.647	240.00	2.17	.01520243	.00002912	.00002572
40	12	5.072	.603	240.00	2.17	.01164383	.00002231	.00002572

REVISED STRUCTURE DEFLECTION AT POINT OF UNIT LOAD 1.92000 (IN)

REVISED STRUCTURE DEFLECTION INFLUENCE PARAMETER .00073942

STRUCTURAL STEEL VOLUME 94375.49 (IN3)

DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	351.972	5.625	144.00	47.09	.20876608	.00003079	.00002572
2	1	273.526	4.875	144.00	47.09	.14060540	.00002074	.00002572
3	2	201.639	4.125	144.00	29.43	.14035199	.00003312	.00002572
4	2	138.273	3.375	144.00	29.43	.07874667	.00001858	.00002572
5	3	85.446	2.625	144.00	14.40	.07734309	.00003730	.00002526
6	3	44.936	1.875	144.00	14.40	.02905533	.00001401	.00002526
7	4	17.748	1.124	144.00	8.23	.01203305	.00001015	.00000516
8	4	3.804	.392	144.00	8.23	.00090079	.00000076	.00000516
9	1	-351.153	-5.625	144.00	47.09	.20826010	.00003071	.00002572
10	1	-272.099	-4.875	144.00	47.09	.13987206	.00002063	.00002572
11	2	-199.986	-4.125	144.00	29.43	.13920135	.00003285	.00002572
12	2	-136.227	-3.375	144.00	29.43	.07758109	.00001831	.00002572
13	3	-83.304	-2.625	144.00	14.40	.07540475	.00003636	.00002526
14	3	-42.814	-1.875	144.00	14.40	.02767967	.00001335	.00002526
15	4	-16.002	-1.126	144.00	8.23	.01087381	.00000918	.00000516
16	4	-2.946	-.358	144.00	8.23	.00063547	.00000054	.00000516
17	0	-3.003	.000	192.00	10.30	-.00000000	-.00000000	0.00000000
18	0	-2.447	-.000	192.00	10.30	.00000000	.00000000	0.00000000
19	0	-4.284	.000	192.00	10.30	-.00000000	-.00000000	0.00000000
20	0	-4.707	-.000	192.00	10.30	.00000004	.00000000	0.00000000
21	0	-6.157	.000	192.00	10.30	-.00000056	-.00000000	0.00000000
22	0	-6.421	-.002	192.00	10.30	.00000638	.00000000	0.00000000
23	0	-7.264	.022	192.00	10.30	-.00010072	-.00000005	0.00000000
24	0	-3.928	-.477	192.00	10.30	.00120358	.00000061	0.00000000
25	5	-69.120	-.625	240.00	7.57	.04720752	.00002597	.00002572
26	5	67.755	.625	240.00	7.57	.04627487	.00002546	.00002572
27	6	-64.001	-.625	240.00	7.26	.04562667	.00002620	.00002572
28	6	61.624	.625	240.00	7.26	.04393162	.00002523	.00002572
29	7	-58.565	-.625	240.00	6.92	.04375632	.00002634	.00002572
30	7	55.810	.625	240.00	6.92	.04169786	.00002510	.00002572
31	8	-50.455	-.625	240.00	6.39	.04082961	.00002662	.00002572
32	8	47.045	.625	240.00	6.39	.03806914	.00002482	.00002572
33	9	-41.160	-.625	240.00	5.74	.03705999	.00002688	.00002572
34	9	37.590	.625	240.00	5.74	.03384756	.00002455	.00002572
35	10	-29.893	-.625	240.00	4.85	.03185801	.00002734	.00002572
36	10	26.357	.625	240.00	4.85	.02807094	.00002409	.00002572
37	11	-18.330	-.623	240.00	3.76	.02512660	.00002784	.00002571
38	11	15.420	.627	240.00	3.76	.02128170	.00002358	.00002571
39	12	-6.340	-.654	240.00	2.17	.01578069	.00003023	.00002578
40	12	4.910	.596	240.00	2.17	.01113266	.00002133	.00002578

STRUCTURE DEFLECTION AT POINT OF UNIT LOAD 1.91999 (IN)

STRUCTURE DEFLECTION INFLUENCE PARAMETER .00073951

DEFLECTION CRITERION SATISFIED

PROGRAM WTD1 RESULTS

	Initial	Revised
Structure steel volume (in^3)	77105.3	94375.5
Structure deflection (in)	2.54002	1.91999
Structure defl. infl. parameter	0.00132	0.00074

Member Modification

		Area (in^2)			
Member	Type	Orig.	Rev.	New Section	Area (in^2)
1,2,9,10	Column	32.70	47.09	W 12 x 161	47.4
3,4,11,12	Column	23.2	29.43	W 12 x 106	31.2
25,26	Bracing	6.48	7.57	L 8 x 8 x 1/2	7.75
27,28	Bracing	5.86	7.26	L 8 x 6 x 9/16	7.56
29,30	Bracing	5.31	6.92	L 6 x 6 x 5/8	7.11
31,32	Bracing	4.75	6.39	L 6 x 6 x 9/16	6.43
33,34	Bracing	4.30	5.74	L 6 x 6 x 1/2	5.75
35,36	Bracing	3.09	4.85	L 6 x 6 x 7/16	5.06
37,38	Bracing	2.09	3.76	L 5 x 5 x 7/16	4.18
39,40	Bracing	1.56	2.17	L 4 x 4 x 5/16	2.4

Final steel volume = 96089.3 in^3

Volume increase = 24.6%

Deflection decrease = 32.3%

Stiffness increase = 43.94%

Final member sizes for Design Example 2 are summarized in Fig. C.9. Note that although the truss is statically indeterminate, the optimization process converged very quickly.

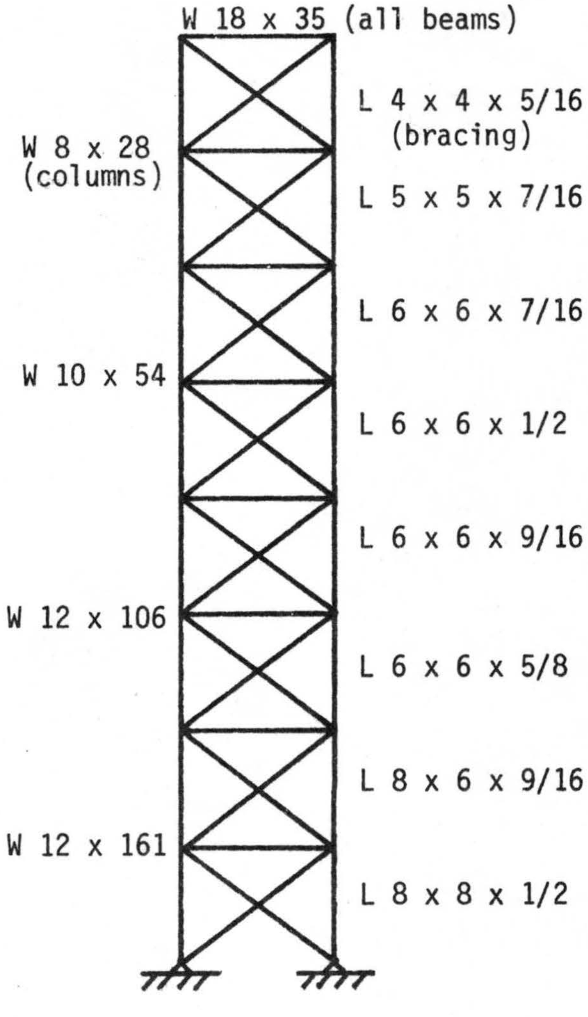

Fig. C.9
Final member sizes
Design Example 2.

C.3 DESIGN EXAMPLE 3

Design Example 3 involves the optimization and subsequent analysis of the one-bay eight-story K-braced frame shown in Fig. C.10. This problem was initially designed for gravity loads only, optimized for drift control by Program WTD1, and then analyzed for strength considerations under combined lateral and gravity loads using a modified version of the original program.

Design Examples 2 and 3 are purposely assigned the same structural dimensions (see Design Example 2) and the same series of design loads. At the conclusion of this design, the final structural steel volumes for the two examples will be compared, indicating the relative efficiencies of the braced frames considering the assumptions and approximations employed in each design.

In Design Example 2, all members that were stress controlled by the combined gravity-wind loading case were substantially modified by Program WTD1. Many member areas were increased by more than 100% over their original strength design areas. For this reason, the K-braced frame in Design Example 3 was initially designed for dead and live loads, and then optimized for drift control using Program WTD1. The revised member areas were then checked for strength considerations using Program PTA1 (Plane Truss Analysis 1), a modified version of the original program. A listing of Program PTA1 and the modifications required to convert Program WTD1 to Program PTA1 follows this design example.

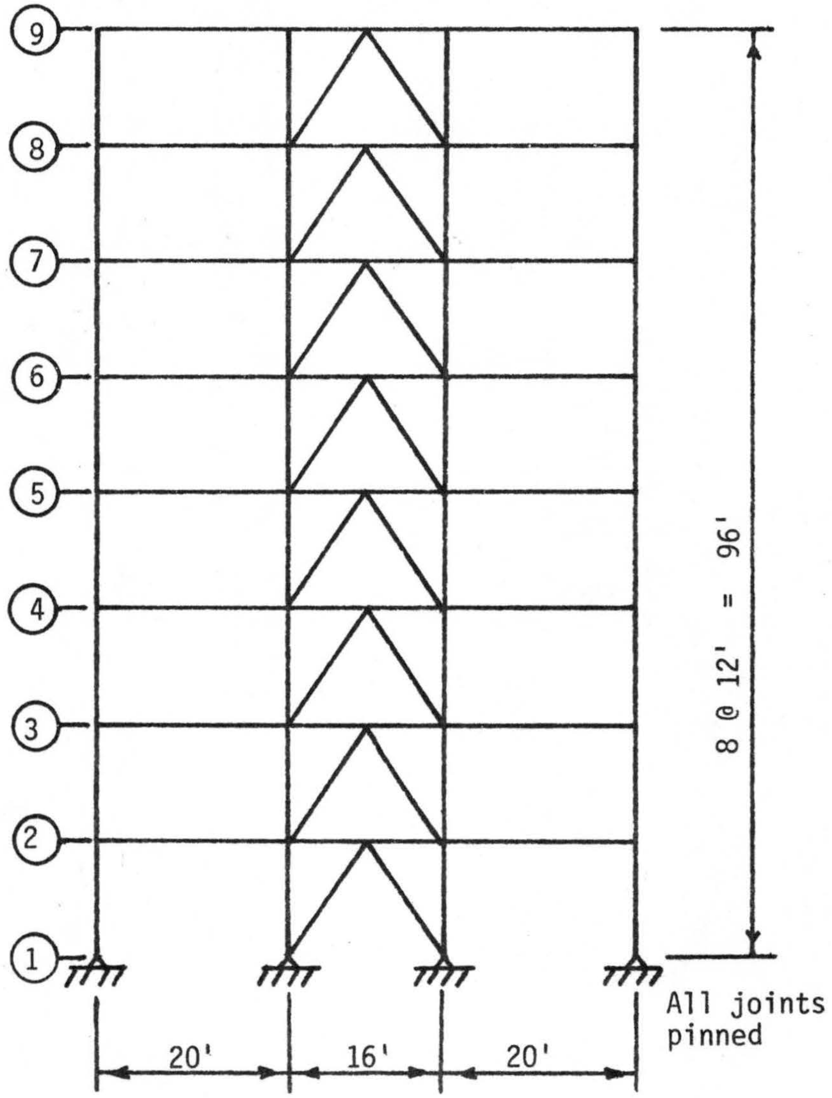

Fig. C.10

K - braced framing system Design Example 3

GRAVITY LOAD DESIGN

<u>Loads</u>

Live load: 80 psf all levels, including roof

Dead load: 50 psf all levels.

Beam Design

Assume, as in Design Examples 1 and 2, that the beams and girders located perpendicular to the braced frames span 25 ft. Assume for design purposes that each beam in the braced frame is continuous over the interior support ("two-span continuous beam").

M_{max} occurs over the interior support and is:

$$1/8\ (w)(L^2) = 26 \text{ k-ft}$$

Select W12 with increased area to recognize axial forces from wind loading.

W 12 x 16.5, M_R = 35 k-ft, A = 4.87 in^2

Column Design

Assume the following trial members:

Level	P_{max} (k)	Trial column	A(in^2)
7-9	100.75	W 8 x 24	7.06
5-7	217.75	W 10 x 39	11.50
3-5	334.75	W 12 x 53	15.60
1-3	451.75	W 12 x 65	19.10

Note that several of these columns are inadequate and will have to be increased in either the optimization routine or in the final design check.

Bracing Design

Maximum bracing member load from floor beam = 19.53 k.

Assume a wide flange section as areas necessary to limit excessive lateral drift may become large.

W 6 x 12, A = 3.54 in^2

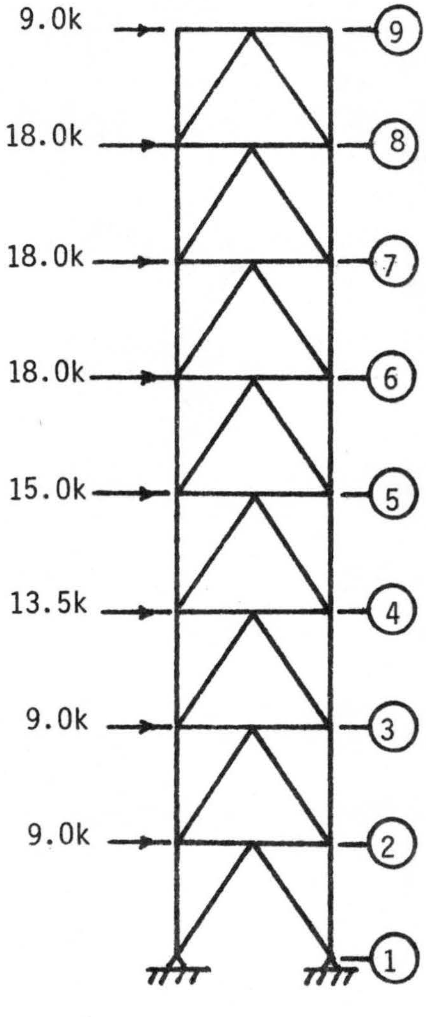

Fig. C.11
Full wind forces
Design Example 3

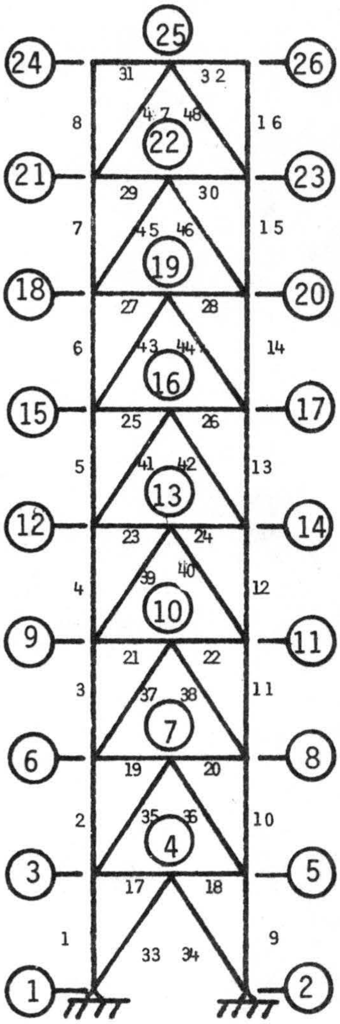

Fig. C.12
Structure model
Design example 3

Program WTD1 Output - Example 3

DESIGN EXAMPLE 3 -- 1 BAY 8 STORY K BRACED TRUSS

NUMBER OF JOINTS = 26

NUMBER OF MEMBERS = 48

JOINT	X(IN)	Y(IN)	XTFIX	YTFIX
1	0.00	0.00	1	1
2	192.00	0.00	1	1
3	0.00	144.00	0	0
4	96.00	144.00	0	0
5	192.00	144.00	0	0
6	0.00	288.00	0	0
7	96.00	288.00	0	0
8	192.00	288.00	0	0
9	0.00	432.00	0	0
10	96.00	432.00	0	0
11	192.00	432.00	0	0
12	0.00	576.00	0	0
13	96.00	576.00	0	0
14	192.00	576.00	0	0
15	0.00	720.00	0	0
16	96.00	720.00	0	0
17	192.00	720.00	0	0
18	0.00	864.00	0	0
19	96.00	864.00	0	0
20	192.00	864.00	0	0
21	0.00	1008.00	0	0
22	96.00	1008.00	0	0
23	192.00	1008.00	0	0
24	0.00	1152.00	0	0
25	96.00	1152.00	0	0
26	192.00	1152.00	0	0

MEMBER	JEND	KEND	L(IN)	A(IN2)	E(KSI)	IDENT
1	1	3	144.00	19.10	29000.00	1
2	3	6	144.00	19.10	29000.00	1
3	6	9	144.00	15.60	29000.00	2
4	9	12	144.00	15.60	29000.00	2
5	12	15	144.00	11.50	29000.00	3
6	15	18	144.00	11.50	29000.00	3
7	18	21	144.00	7.06	29000.00	4
8	21	24	144.00	7.06	29000.00	4
9	2	5	144.00	19.10	29000.00	1
10	5	8	144.00	19.10	29000.00	1
11	8	11	144.00	15.60	29000.00	2
12	11	14	144.00	15.60	29000.00	2
13	14	17	144.00	11.50	29000.00	3
14	17	20	144.00	11.50	29000.00	3

15	20	23	144.00	7.06	29000.00	4
16	23	26	144.00	7.06	29000.00	4
17	3	4	96.00	4.87	29000.00	5
18	4	5	96.00	4.87	29000.00	5
19	6	7	96.00	4.87	29000.00	6
20	7	8	96.00	4.87	29000.00	6
21	9	10	96.00	4.87	29000.00	7
22	10	11	96.00	4.87	29000.00	7
23	12	13	96.00	4.87	29000.00	8
24	13	14	96.00	4.67	29000.00	8
25	15	16	96.00	4.87	29000.00	9
26	16	17	96.00	4.87	29000.00	9
27	18	19	96.00	4.87	29000.00	10
28	19	20	96.00	4.87	29000.00	10
29	21	22	96.00	4.67	29000.00	11
30	22	23	96.00	4.87	29000.00	11
31	24	25	96.00	4.87	29000.00	12
32	25	26	96.00	4.87	29000.00	12
33	1	4	173.07	3.54	29000.00	13
34	4	2	173.07	3.54	29000.00	13
35	3	7	173.07	3.54	29000.00	14
36	7	5	173.07	3.54	29000.00	14
37	6	10	173.07	3.54	29000.00	15
38	10	8	173.07	3.54	29000.00	15
39	9	13	173.07	3.54	29000.00	16
40	13	11	173.07	3.54	29000.00	16
41	12	16	173.07	3.54	29000.00	17
42	16	14	173.07	3.54	29000.00	17
43	15	19	173.07	3.54	29000.00	18
44	19	17	173.07	3.54	29000.00	18
45	18	22	173.07	3.54	29000.00	19
46	22	20	173.07	3.54	29000.00	19
47	21	25	173.07	3.54	29000.00	20
48	25	23	173.07	3.54	29000.00	20

STRUCTURAL STEEL VOLUME 47969.56 (IN3)

APPLIED JOINT LOADS

JOINT	X FORCE(K)	Y FORCE(K)
3	9.000	0.000
6	9.000	0.000
9	13.500	0.000
12	15.000	0.000
15	18.000	0.000
18	18.000	0.000
21	15.000	0.000
24	9.000	0.000

APPLIED JOINT LOADS

JOINT	X FORCE(K)	Y FORCE(K)
3	9.000	0.000
6	9.000	0.000
9	13.500	0.000
12	15.000	0.000
15	18.000	0.000
18	18.000	0.000
21	15.000	0.000
24	9.000	0.000

APPLIED UNIT LOAD

JOINT	X FORCE	Y FORCE
24	1.000	0.000

MAXIMUM ALLOWABLE DEFLECTION 1.92000 (IN)

DEFLECTION CONVERGENCE LIMIT 0.00000 (IN)

DETERMINATE STRUCTURAL ANALYSIS

DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	310.500	5.250	144.00	19.10	.42379130	.00015408	.00012705
2	1	235.125	4.500	144.00	19.10	.27506951	.00010001	.00012705
3	2	166.500	3.750	144.00	15.60	.19874005	.00008847	.00006719
4	2	108.000	3.000	144.00	15.60	.10312997	.00004591	.00006719
5	3	60.750	2.250	144.00	11.50	.05901949	.00003564	.00002310
6	3	27.000	1.500	144.00	11.50	.01748726	.00001056	.00002310
7	4	6.750	.750	144.00	7.06	.00356061	.00000350	.00000175
8	4	-.000	-.000	144.00	7.06	.00000000	.00000000	.00000175
9	1	-310.500	-5.250	144.00	19.10	.42379130	.00015408	.00012705
10	1	-235.125	-4.500	144.00	19.10	.27506951	.00010001	.00012705
11	2	-166.500	-3.750	144.00	15.60	.19874005	.00008847	.00006719
12	2	-108.000	-3.000	144.00	15.60	.10312997	.00004591	.00006719
13	3	-60.750	-2.250	144.00	11.50	.05901949	.00003564	.00002310
14	3	-27.000	-1.500	144.00	11.50	.01748726	.00001056	.00002310
15	4	-6.750	-.750	144.00	7.06	.00356061	.00000350	.00000175
16	4	-.000	-.000	144.00	7.06	.00000000	.00000000	.00000175
17	5	-59.250	-.500	96.00	4.87	.02013736	.00004307	.00003960
18	5	50.250	.500	96.00	4.87	.01707852	.00003653	.00003960
19	6	-54.750	-.500	96.00	4.87	.01860794	.00003980	.00003653
20	6	45.750	.500	96.00	4.87	.01554910	.00003326	.00003653
21	7	-52.500	-.500	96.00	4.87	.01784323	.00003817	.00003326
22	7	39.000	.500	96.00	4.87	.01325497	.00002835	.00003326

23	8	-46.500	-.500	96.00	4.87	.01580401	.00003380	.00002835
24	8	31.500	.500	96.00	4.87	.01070594	.00002290	.00002835
25	9	-40.500	-.500	96.00	4.87	.01376478	.00002944	.00002290
26	9	22.500	.500	96.00	4.87	.00764710	.00001636	.00002290
27	10	-31.500	-.500	96.00	4.87	.01070594	.00002290	.00001636
28	10	13.500	.500	96.00	4.87	.00458826	.00000981	.00001636
29	11	-22.500	-.500	96.00	4.87	.00764710	.00001636	.00000981
30	11	4.500	.500	96.00	4.87	.00152942	.00000327	.00000981
31	12	-9.000	-1.000	96.00	4.87	.00611768	.00001309	.00000654
32	12	.000	.000	96.00	4.87	.00000000	.00000000	.00000654
33	13	98.702	.901	173.07	3.54	.14998545	.00024481	.00024481
34	13	-98.702	-.901	173.07	3.54	.14998545	.00024481	.00024481
35	14	90.569	.901	173.07	3.54	.13765788	.00022469	.00022469
36	14	-90.569	-.901	173.07	3.54	.13765788	.00022469	.00022469
37	15	82.477	.901	173.07	3.54	.12533031	.00020457	.00020457
38	15	-82.477	-.901	173.07	3.54	.12533031	.00020457	.00020457
39	16	70.308	.901	173.07	3.54	.10683895	.00017439	.00017439
40	16	-70.308	-.901	173.07	3.54	.10683895	.00017439	.00017439
41	17	56.767	.901	173.07	3.54	.08629300	.00014085	.00014085
42	17	-56.767	-.901	173.07	3.54	.08629300	.00014085	.00014085
43	18	40.562	.901	173.07	3.54	.06163786	.00010061	.00010061
44	18	-40.562	-.901	173.07	3.54	.06163786	.00010061	.00010061
45	19	24.337	.901	173.07	3.54	.03698271	.00006036	.00006036
46	19	-24.337	-.901	173.07	3.54	.03698271	.00006036	.00006036
47	20	8.112	.901	173.07	3.54	.01232757	.00002012	.00002012
48	20	-8.112	-.901	173.07	3.54	.01232757	.00002012	.00002012

STRUCTURE DEFLECTION AT POINT OF UNIT LOAD 3.77669 (IN)

STRUCTURE DEFLECTION INFLUENCE PARAMETER .00360427

REVISED DEFLECTION CALCULATIONS

MEMBER	IDENT	REAL LOAD(K)	UNIT LOAD	L(IN)	A(IN2)	DEF(IN)	DEF INF PAR	IDENT INF PAR
1	1	310.500	5.250	144.00	41.85	.19342995	.00003210	.00002647
2	1	235.125	4.500	144.00	41.85	.12554925	.00002083	.00002647
3	2	166.500	3.750	144.00	24.86	.12473492	.00003485	.00002647
4	2	108.000	3.000	144.00	24.86	.06472731	.00001808	.00002647
5	3	60.750	2.250	144.00	11.50	.05901949	.00003564	.00002310
6	3	27.000	1.500	144.00	11.50	.01748726	.00001056	.00002310
7	4	6.750	.750	144.00	7.06	.00358061	.00000350	.00000175
8	4	-.000	-.000	144.00	7.06	.00000000	.00000000	.00000175
9	1	-310.500	-5.250	144.00	41.85	.19342995	.00003210	.00002647
10	1	-235.125	-4.500	144.00	41.85	.12554925	.00002083	.00002647
11	2	-166.500	-3.750	144.00	24.86	.12473492	.00003485	.00002647
12	2	-108.000	-3.000	144.00	24.86	.06472731	.00001808	.00002647
13	3	-60.750	-2.250	144.00	11.50	.05901949	.00003564	.00002310

14	3	-27.000	-1.500	144.00	11.50	.01748726	.00001056	.00002310
15	4	-6.750	-.750	144.00	7.06	.00356061	.00000350	.00000175
16	4	-.000	-.000	144.00	7.06	.00000000	.00000000	.00000175
17	5	-59.250	-.500	96.00	5.97	.01642130	.00002864	.00002647
18	5	50.250	.500	96.00	5.97	.01392693	.00002429	.00002647
19	6	-54.750	-.500	96.00	5.72	.01583899	.00002884	.00002647
20	6	45.750	.500	96.00	5.72	.01323532	.00002410	.00002647
21	7	-52.500	-.500	96.00	5.46	.01591751	.00003037	.00002647
22	7	39.000	.500	96.00	5.46	.01182443	.00002256	.00002647
23	8	-46.500	-.500	96.00	5.04	.01526975	.00003156	.00002647
24	8	31.500	.500	96.00	5.04	.01034403	.00002138	.00002647
25	9	-40.500	-.500	96.00	4.87	.01376478	.00002944	.00002290
26	9	22.500	.500	96.00	4.87	.00764710	.00001636	.00002290
27	10	-31.500	-.500	96.00	4.87	.01070594	.00002290	.00001636
28	10	13.500	.500	96.00	4.87	.00458826	.00000981	.00001636
29	11	-22.500	-.500	96.00	4.87	.00764710	.00001636	.00000981
30	11	4.500	.500	96.00	4.87	.00152942	.00000327	.00000981
31	12	-9.000	-1.000	96.00	4.87	.00611768	.00001309	.00000654
32	12	.000	.000	96.00	4.87	.00000000	.00000000	.00000654
33	13	98.702	.901	173.07	10.77	.04931588	.00002647	.00002647
34	13	-98.702	-.901	173.07	10.77	.04931588	.00002647	.00002647
35	14	90.589	.901	173.07	10.31	.04724575	.00002647	.00002647
36	14	-90.589	-.901	173.07	10.31	.04724575	.00002647	.00002647
37	15	82.477	.901	173.07	9.84	.04508066	.00002647	.00002647
38	15	-82.477	-.901	173.07	9.84	.04508066	.00002647	.00002647
39	16	70.308	.901	173.07	9.09	.04162239	.00002647	.00002647
40	16	-70.308	-.901	173.07	9.09	.04162239	.00002647	.00002647
41	17	56.787	.901	173.07	8.17	.03740675	.00002647	.00002647
42	17	-56.787	-.901	173.07	8.17	.03740675	.00002647	.00002647
43	18	40.562	.901	173.07	6.90	.03161447	.00002647	.00002647
44	18	-40.562	-.901	173.07	6.90	.03161447	.00002647	.00002647
45	19	24.337	.901	173.07	5.35	.02448847	.00002647	.00002647
46	19	-24.337	-.901	173.07	5.35	.02448847	.00002647	.00002647
47	20	8.112	.901	173.07	3.54	.01232757	.00002012	.00002012
48	20	-8.112	-.901	173.07	3.54	.01232757	.00002012	.00002012

REVISED STRUCTURE DEFLECTION AT POINT OF UNIT LOAD 1.92000 (IN)

REVISED STRUCTURE DEFLECTION INFLUENCE PARAMETER .00104489

STRUCTURAL STEEL VOLUME 79252.07 (IN3)

DEFLECTION CRITERION SATISFIED

PROGRAM WTD1 REVISIONS OF MEMBERS

Member	Type	Revised Area (in^2)	New Section	Section Area (in^2)	P_{ALLOW}(k)
1,2,9,10	Column	41.85	W 14 x 150	44.1	860.0
3,4,11,12	Column	24.86	W 12 x 85	25.0	465.0
17,18	Beam	5.97	W 12 x 22	6.47	139.8
19,20	Beam	5.72	W 12 x 22	6.47	139.8
21,22	Beam	5.46	W 12 x 19	5.59	120.7
23,24	Beam	5.04	W 12 x 19	5.59	120.7
33,34	Bracing	10.77	W 8 x 40	11.8	174.6
35,36	Bracing	10.31	W 8 x 35	10.30	152.0
37,38	Bracing	9.84	W 8 x 35	10.30	152.0
39,40	Bracing	9.09	W 8 x 31	9.12	133.5
41,42	Bracing	8.17	W 8 x 28	8.23	99.6
43,44	Bracing	6.90	W 6 x 25	7.35	82.6
45,46	Bracing	5.35	W 6 x 20	5.88	65.0

Program PTA1 Output - Example 3

DESIGN EXAMPLE 3 -- 1 BAY 8 STORY K BRACED TRUSS

NUMBER OF JOINTS = 26

NUMBER OF MEMBERS = 48

JOINT	X(IN)	Y(IN)	XTFIX	YTFIX
1	0.00	0.00	1	1
2	192.00	0.00	1	1
3	0.00	144.00	0	0
4	96.00	144.00	0	0
5	192.00	144.00	0	0
6	0.00	288.00	0	0
7	96.00	288.00	0	0
8	192.00	288.00	0	0
9	0.00	432.00	0	0
10	96.00	432.00	0	0
11	192.00	432.00	0	0
12	0.00	576.00	0	0
13	96.00	576.00	0	0
14	192.00	576.00	0	0
15	0.00	720.00	0	0
16	96.00	720.00	0	0
17	192.00	720.00	0	0
18	0.00	864.00	0	0
19	96.00	864.00	0	0
20	192.00	864.00	0	0
21	0.00	1008.00	0	0
22	96.00	1008.00	0	0
23	192.00	1008.00	0	0
24	0.00	1152.00	0	0
25	96.00	1152.00	0	0
26	192.00	1152.00	0	0

MEMBER	JEND	KEND	L(IN)	A(IN2)	E(KSI)	IDENT
1	1	3	144.00	44.10	29000.00	1
2	3	6	144.00	44.10	29000.00	1
3	6	9	144.00	25.00	29000.00	2
4	9	12	144.00	25.00	29000.00	2
5	12	15	144.00	11.50	29000.00	3
6	15	18	144.00	11.50	29000.00	3
7	18	21	144.00	7.06	29000.00	4
8	21	24	144.00	7.06	29000.00	4
9	2	5	144.00	44.10	29000.00	1
10	5	8	144.00	44.10	29000.00	1
11	8	11	144.00	25.00	29000.00	2
12	11	14	144.00	25.00	29000.00	2
13	14	17	144.00	11.50	29000.00	3
14	17	20	144.00	11.50	29000.00	3

15	20	23	144.00	7.06	29000.00	4
16	23	26	144.00	7.06	29000.00	4
17	3	4	96.00	6.47	29000.00	5
18	4	5	96.00	6.47	29000.00	5
19	6	7	96.00	6.47	29000.00	6
20	7	8	96.00	6.47	29000.00	6
21	9	10	96.00	5.59	29000.00	7
22	10	11	96.00	5.59	29000.00	7
23	12	13	96.00	5.59	29000.00	8
24	13	14	96.00	5.59	29000.00	8
25	15	16	96.00	4.87	29000.00	9
26	16	17	96.00	4.87	29000.00	9
27	18	19	96.00	4.87	29000.00	10
28	19	20	96.00	4.87	29000.00	10
29	21	22	96.00	4.87	29000.00	11
30	22	23	96.00	4.87	29000.00	11
31	24	25	96.00	4.87	29000.00	12
32	25	26	96.00	4.87	29000.00	12
33	1	4	173.07	11.80	29000.00	13
34	4	2	173.07	11.80	29000.00	13
35	3	7	173.07	10.30	29000.00	14
36	7	5	173.07	10.30	29000.00	14
37	6	10	173.07	10.30	29000.00	15
38	10	8	173.07	10.30	29000.00	15
39	9	13	173.07	9.12	29000.00	16
40	13	11	173.07	9.12	29000.00	16
41	12	16	173.07	8.23	29000.00	17
42	16	14	173.07	8.23	29000.00	17
43	15	19	173.07	7.35	29000.00	18
44	19	17	173.07	7.35	29000.00	18
45	18	22	173.07	5.88	29000.00	19
46	22	20	173.07	5.88	29000.00	19
47	21	25	173.07	3.54	29000.00	20
48	25	23	173.07	3.54	29000.00	20

STRUCTURAL STEEL VOLUME 81888.12 (IN3)

APPLIED JOINT LOADS

JOINT	X FORCE(K)	Y FORCE(K)
3	0.000	-45.000
4	0.000	-32.500
5	0.000	-45.000
6	0.000	-45.000
7	0.000	-32.500
8	0.000	-45.000
9	0.000	-44.000
10	0.000	-32.500
11	0.000	-44.000
12	0.000	-44.000
13	0.000	-32.500

14	0.000	-44.000
15	0.000	-43.000
16	0.000	-32.500
17	0.000	-43.000
18	0.000	-43.000
19	0.000	-32.500
20	0.000	-43.000
21	0.000	-42.500
22	0.000	-32.500
23	0.000	-42.500
24	0.000	-42.500
25	0.000	-32.500
26	0.000	-42.500

APPLIED JOINT LOADS

JOINT	X FORCE(K)	Y FORCE(K)
3	6.750	-33.750
4	0.000	-24.380
5	0.000	-33.750
6	6.750	-33.750
7	0.000	-24.380
8	0.000	-33.750
9	10.130	-33.000
10	0.000	-24.380
11	0.000	-33.000
12	11.250	-33.000
13	0.000	-24.380
14	0.000	-33.000
15	13.500	-32.250
16	0.000	-24.380
17	0.000	-32.250
18	13.500	-32.250
19	0.000	-24.380
20	0.000	-32.250
21	13.500	-31.880
22	0.000	-24.380
23	0.000	-31.880
24	6.750	-31.880
25	0.000	-24.380
26	0.000	-31.880

MEMBER AXIAL FORCE CALCULATIONS

MEMBER	A(IN2)	CASE 1	CASE 2
1	44.10	-462.750	-114.207
2	44.10	-401.500	-124.802
3	25.00	-340.250	-130.335
4	25.00	-280.000	-129.020
5	11.50	-219.750	-119.267
6	11.50	-160.500	-100.140
7	7.06	-101.250	-70.887
8	7.06	-42.500	-31.880
9	44.10	-462.750	-579.973
10	44.10	-401.500	-477.498
11	25.00	-340.250	-380.085
12	25.00	-280.000	-291.020
13	11.50	-219.750	-210.393
14	11.50	-160.500	-140.640
15	7.06	-101.250	-81.013
16	7.06	-42.500	-31.880
17	6.47	10.833	-36.313
18	6.47	10.833	45.817
19	6.47	10.833	-32.938
20	6.47	10.833	42.442
21	5.59	10.833	-31.253
22	5.59	10.833	37.377
23	5.59	10.833	-26.748
24	5.59	10.833	31.752
25	4.87	10.833	-22.248
26	4.87	10.833	25.002
27	4.87	10.833	-15.498
28	4.87	10.833	18.252
29	4.87	10.833	-8.748
30	4.87	10.833	11.502
31	4.87	0.000	-6.750
32	4.87	-.000	.000
33	11.80	-19.530	59.380
34	11.80	-19.530	-88.682
35	10.30	-19.530	53.296
36	10.30	-19.530	-82.597
37	10.30	-19.530	47.212
38	10.30	-19.530	-76.513
39	9.12	-19.530	38.081
40	9.12	-19.530	-67.382
41	8.23	-19.530	27.940
42	8.23	-19.530	-57.241
43	7.35	-19.530	15.771
44	7.35	-19.530	-45.072
45	5.88	-19.530	3.603
46	5.88	-19.530	-32.904
47	3.54	-19.530	-8.566
48	3.54	-19.530	-20.735

PROGRAM PTA1 ANALYSIS

Member	Type	$P_{MAX}(k)$	O.K.?	New Section	Area (in^2)
1,2,9,10	Column	580.0	O.K.	---	---
3,4,11,12	Column	380.1	O.K.	---	---
5,6,13,14	Column	219.8	N.G.	W 10 x 49	14.4
7,8,15,16	Column	101.3	O.K	---	---
17,18	Beam	36.3	O.K.(0.77)*	---	---
19,20	Beam	32.9	O.K.(0.75)	---	---
21,22	Beam	31.3	O.K.(0.87)	---	---
23,24	Beam	26.7	O.K.(0.83)	---	---
25,26	Beam	22.2	O.K.(0.95)	---	---
27,28	Beam	15.5	O.K.(0.89)	---	---
29,30	Beam	8.7	O.K.(0.83)	---	---
31,32	Beam	6.8	O.K.(0.81)	---	---
33,34	Bracing	88.7	O.K.	---	---
35,36	Bracing	82.6	O.K.	---	---
37,38	Bracing	76.5	O.K.	---	---
39,40	Bracing	67.4	O.K.	---	---
41,42	Bracing	57.2	O.K.	---	---
43,44	Bracing	45.1	O.K.	---	---
45,46	Bracing	32.9	O.K.	---	---
47,48	Bracing	20.7	N.G.	W 6 x 16	4.72

*Values from interaction equation, AISC Eq. 1.6-1b.

Beams assumed fully laterally braced by floor system.

PROGRAM WTD1 AND PROGRAM PTA1 RESULTS

Drift design:

		Initial	Revised
Structure steel volume (in^3)		47960.6	79252.1
Structure deflection (in)		3.77669	1.92000
Structure delf. infl. parameter		0.00360	0.00104
Structure deflection decrease	=	49.16%	
Structural stiffness increase	=	96.70%	

Strength check:

Structure steel volume	=	81888.1 in^3
Final design volume	=	83967.4 in^3
Structural steel volume increase (from original design to final design)	=	75.08%

In Design Example 3, an increase in steel volume of 75.08% above that required for the somewhat inadequate original design produced the necessary 96.07% increase in structural stiffness.

The final volumes of structural steel required for Design Examples 2 and 3 are presented below:

Design Example 2 steel volume	=	96089.3 in^3
Design Example 3 steel volume	=	83967.4 in^3

A comparison of these two values indicates that the K-braced frame, shown in Design Example 3, is the more efficient system for the configuration and design loads assumed.

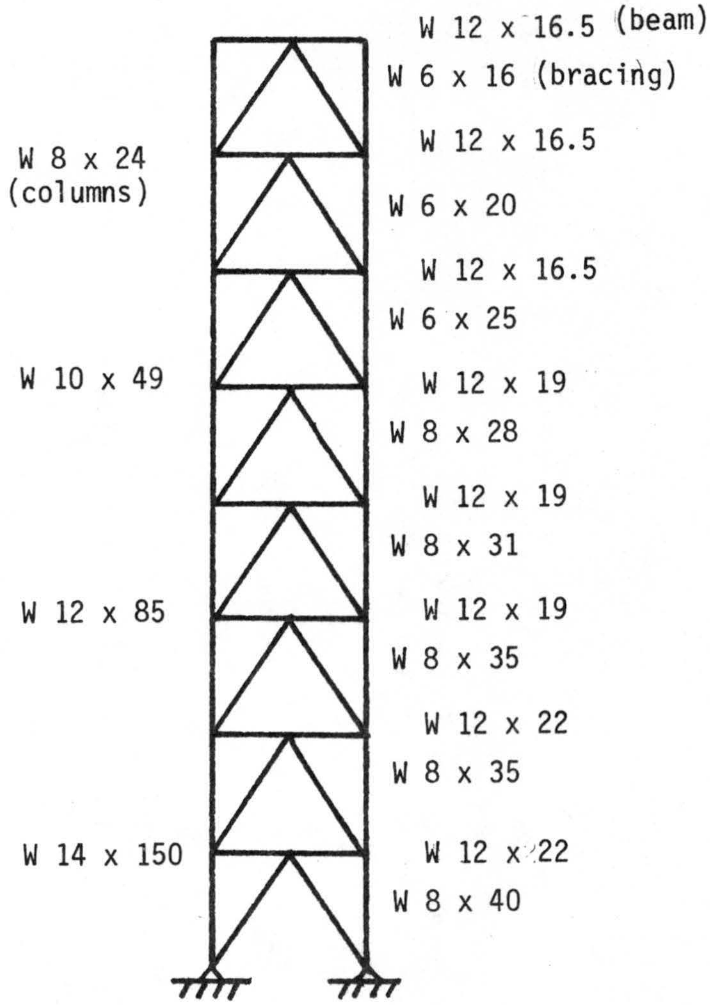

Fig. C.13

Final members

Design Example 3.

9) remove SUBROUTINE DEFL and SUBROUTINE GROUP entirely.

A listing of Program PTA1 follows.

PROGRAM PTA1

Program PTA1 (Plane Truss Analysis 1) is one of many possible structural analysis programs derivable from Program WTD1. Program PTA1 is capable of analyzing braced frames for two individual loading cases. Few data file revisions are required to execute this program; the data for the second loading case is placed immediately behind the first loading case and the deflection information statement must be removed.

Modifications to the original program are:

1) revise the PROGRAM "NAME" statement contained in line 1,
2) remove the COMMON/INFORME/ and INFORMF/ statements, lines 6 and 7, and replace them with a DIMENSION statement reserving core space for RNN, RN, and IDENT.
3) remove the NUMID statements in lines 44 and 56,
4) remove statements concerning the array PA, lines 69 and 107, and the entire unit load section, lines 92 through 108,
5) place the applied joint loads section in a DO loop extending from line 78 to line 91, to cycle through the two loading cases, and change the AJ statement (line 88) to reflect the DO parameter,
6) remove the deflection criteria section, lines 109 through 125,
7) remove the deflection contribution section, lines 212 through 308,
8) revise the WRITE statements and required to receive the desired output, and

Listing of Program PTA1

```
      PROGRAM PTA1 (INPUT,OUTPUT,TAPE1=INPUT,TAPE2=OUTPUT)
      COMMON /INFORMA/ RT(4,4),SINT(100),COST(100),RTRANS(4,4)
      COMMON /INFORMB/ ALEN(100),E(100),AA(100),SM(4,4)
      COMMON /INFORMC/ JC(50),KC(50),IN(4)
      COMMON /INFORMD/ S(100,20),AJ(100,2),DJ(100,2)
      DIMENSION ITITLE(8), X(50), Y(50), XTFIX(50), YTFIX(50)
      DIMENSION RNN(100), RN(100), IDENT(100), TEMP(4,4), SMR(4,4)
      DIMENSION AM(400,2), JTL(4), D(4), AMT(4)
      REAL LX,LY,MDEFP,JTL
      INTEGER XTFIX,YTFIX
C
C     STRUCTURE DATA
C
      READ (1,128) ITITLE
      WRITE (2,129) ITITLE
      READ (1,130) NJOINT,NMEM
      WRITE (2,131) NJOINT
      WRITE (2,132) NMEM
      DO 101 I=1,NJOINT
         READ (1,133) JN,X(JN),Y(JN),XTFIX(JN),YTFIX(JN)
  101 CONTINUE
      WRITE (2,134)
      DO 102 I=1,NJOINT
         WRITE (2,135) I,X(I),Y(I),XTFIX(I),YTFIX(I)
  102 CONTINUE
      DO 103 I=1,NMEM
         READ (1,136) M,JC(M),KC(M),AA(M),E(M),IDENT(M)
  103 CONTINUE
      WRITE (2,137)
      VOLUME=0.0
      DO 104 I=1,NMEM
         JJ=JC(I)
         KK=KC(I)
         LX=X(KK)-X(JJ)
         LY=Y(KK)-Y(JJ)
         ALEN(I)=SQRT(LX*LX+LY*LY)
         WRITE (2,138) I,JC(I),KC(I),ALEN(I),AA(I),E(I),IDENT(I)
         COST(I)=LX/ALEN(I)
         SINT(I)=LY/ALEN(I)
         VOLUME=VOLUME+(AA(I)*ALEN(I))
  104 CONTINUE
      WRITE (2,139) VOLUME
C
C     LOADING DATA
C     APPLIED JOINT LOADS
C
      N2=2*NJOINT
      DO 106 I=1,2
         DO 105 J=1,N2
            AJ(J,I)=0.0
  105    CONTINUE
  106 CONTINUE
      DO 109 N=1,2
         READ (1,140) NJTL
         WRITE (2,141)
         WRITE (2,142)
         DO 108 I=1,NJTL
            READ (1,143) JN,(JTL(J),J=1,2)
            WRITE (2,144) JN,(JTL(J),J=1,2)
            J2=2*JN
            IN(1)=J2-1
```

```
      IN(2)=J2
      DO 107 IA=1,2
         MA=IN(IA)
         AJ(MA,N)=AJ(MA,N)+JTL(IA)
  107 CONTINUE
  108 CONTINUE
  109 CONTINUE
C
C     MEMBER AND STRUCTURE STIFFNESS MATRICES
C
      IBAND=0
      DO 111 I=1,N2
         DO 110 J=1,20
            S(I,J)=0.0
  110    CONTINUE
  111 CONTINUE
      DO 114 I=1,NMEM
         CALL FORMRT (I)
         CALL FORMSM (I)
         CALL MULT (RTRANS,4,4,SM,4,4,TEMP)
         CALL MULT (TEMP,4,4,RT,4,4,SMR)
C
C     STIFFNESS MATRIX STORAGE
C
         CALL INDEX (I)
         DO 113 J=1,4
            JJ=IN(J)
            DO 112 K=1,4
               KK=IN(K)
               IF (JJ.GT.KK) GO TO 112
               KK=KK-JJ+1
               IF (KK.GT.IBAND.AND.SMR(J,J).NE.0.0) IBAND=KK
               S(JJ,KK)=S(JJ,KK)+SMR(J,K)
  112       CONTINUE
  113    CONTINUE
  114 CONTINUE
      N4=4*NMEM
      DO 116 I=1,2
         DO 115 J=1,N4
            AM(J,I)=0.0
  115    CONTINUE
  116 CONTINUE
C
C     S AND AJ BOUNDARY CONDITION MODIFICATIONS
C
      RFAC=1.0E+60
      DO 120 I=1,NJOINT
         I2=2*I
         IF (YTFIX(I).EQ.0) GO TO 118
         S(I2,1)=RFAC
         DO 117 J=1,2
            AJ(I2,J)=0.0
  117    CONTINUE
  118    IF (XTFIX(I).EQ.0) GO TO 120
         I1=I2-1
         S(I1,1)=RFAC
         DO 119 J=1,2
            AJ(I1,J)=0.0
  119    CONTINUE
  120 CONTINUE
C
```

```
      CALL SOLVE (NJOINT,IBAND)
C
C     MEMBER END ACTIONS
C
      DO 124 N=1,2
         DO 123 I=1,NMEM
            CALL INDEX (I)
            DO 121 IA=1,4
               M=IN(IA)
               D(IA)=DJ(M,N)
  121       CONTINUE
            CALL FORMSM (I)
            CALL FORMRT (I)
            CALL MULT (SM,4,4,RT,4,4,TEMP)
            CALL MULT (TEMP,4,4,D,4,1,AMT)
            DO 122 IA=1,4
               M=4*(I-1)+IA
               AM(M,N)=AM(M,N)+AMT(IA)
  122       CONTINUE
  123    CONTINUE
  124 CONTINUE
      DO 125 I=1,NMEM
         RNN(I)=0.0
         RN(I)=0.0
  125 CONTINUE
      DO 126 M=1,NMEM
         L=4*(M-1)+3
         RNN(M)=AM(L,1)
         RN(M)=AM(L,2)
  126 CONTINUE
      WRITE (2,145)
      WRITE (2,146)
      DO 127 I=1,NMEM
         WRITE (2,147) I,AA(I),RNN(I),RN(I)
  127 CONTINUE
C
  128 FORMAT (8A10)
  129 FORMAT (1H1,///,21X,8A10)
  130 FORMAT (2I5)
  131 FORMAT (1H0,20X, 19HNUMBER OF JOINTS  =,I5)
  132 FORMAT (1H0,20X, 19HNUMBER OF MEMBERS =,I5)
  133 FORMAT (I5,2F10.2,2I5)
  134 FORMAT (1H0,//,21X,  5HJOINT,5X,  5HX(IN),5X,  5HY(IN),5X,  5HXTFI
     1X,5X,  5HYTFIX,/)
  135 FORMAT (21X,I5,2F10.2,2I10)
  136 FORMAT (3I5,2F10.2,I5)
  137 FORMAT (1H0,//,21X,  6HMEMBER,6X,  4HJEND,6X,  4HKEND,5X,  5HL(IN)
     1,4X,  6HA(IN2),4X,  6HE(KSI),5X,  5HIDENT,/)
  138 FORMAT (21X,I6,2I10,3F10.2,I10)
  139 FORMAT (1H0,20X, 23HSTRUCTURAL STEEL VOLUME,2X,F10.2,  6H (IN3))
  140 FORMAT (I5)
  141 FORMAT (1H0,//,21X, 19HAPPLIED JOINT LOADS)
  142 FORMAT (1H0,20X,  5HJOINT,5X, 10HX FORCE(K),5X, 10HY FORCE(K),/)
  143 FORMAT (I5,2F10.2)
  144 FORMAT (21X,I5,2F15.3)
  145 FORMAT (1H0,//,21X, 31HMEMBER AXIAL FORCE CALCULATIONS)
  146 FORMAT (1H0,20X,  6HMEMBER,5X,  6HA(IN2),7X,  6HCASE 1,7X,  6HCASE
     1 2)
  147 FORMAT (21X,I6,F11.2,2F13.3)
C
      END
```

```
      SUBROUTINE FORMRT (M)
      COMMON /INFORMA/ RT(4,4),SINT(100),COST(100),RTRANS(4,4)
      DO 101 I=1,4
      DO 101 J=1,4
         RT(I,J)=0.0
  101 CONTINUE
      CT=COST(M)
      ST=SINT(M)
      RT(1,1)=CT
      RT(2,2)=CT
      RT(3,3)=CT
      RT(4,4)=CT
      RT(1,2)=ST
      RT(3,4)=ST
      RT(2,1)=-ST
      RT(4,3)=-ST
C
C     ROTATION MATRIX TRANSPOSE
C
      DO 102 I=1,4
      DO 102 J=1,4
         RTRANS(J,I)=RT(I,J)
  102 CONTINUE
      RETURN
C
      END
```

```
      SUBROUTINE MULT (A,MA,NA,B,MB,NB,C)
      DIMENSION A(4,4), B(4,4), C(4,4)
      DO 102 I=1,MA
      DO 102 J=1,NB
         SUM=0.0
         DO 101 L=1,NA
            SUM=SUM+A(I,L)*B(L,J)
  101    CONTINUE
         C(I,J)=SUM
  102 CONTINUE
      RETURN
C
      END
```

```
      SUBROUTINE FORMSM (M)
      COMMON /INFORMB/ ALEN(100),E(100),AA(100),SM(4,4)
      AL=ALEN(M)
      A=(AA(M)*E(M))/AL
      DO 101 I=1,4
      DO 101 J=1,4
        SM(I,J)=0.0
  101 CONTINUE
      SM(1,1)=A
      SM(3,3)=A
      SM(1,3)=-A
      SM(3,1)=-A
      RETURN
C
      END
```

```
      SUBROUTINE INDEX (M)
      COMMON /INFORMC/ JC(50),KC(50),IN(4)
      JJ=JC(M)
      KK=KC(M)
      J2=2*JJ
      K2=2*KK
      IN(2)=J2
      IN(1)=J2-1
      IN(4)=K2
      IN(3)=K2-1
      RETURN
C
      END
```

```
      SUBROUTINE SOLVE (NJOINT,IBAND)
      COMMON /INFORMD/ S(100,20),AJ(100,2),DJ(100,2)
C
C     GAUSSIAN ELIMINATION
C
      NEQ=2*NJOINT
      NEQ1=NEQ-1
      DO 105 I=1,NEQ1
         DIAG=S(I,1)
         JEND=NEQ-I+1
         IF (JEND.GT.IBAND) JEND=IBAND
         DO 103 J=2,JEND
            JROW=I+J-1
            FAC=S(I,J)/DIAG
            JCOL=0
            DO 101 K=J,JEND
               JCOL=JCOL+1
               S(JROW,JCOL)=S(JROW,JCOL)-S(I,K)*FAC
  101       CONTINUE
            DO 102 K=1,2
               AJ(JROW,K)=AJ(JROW,K)-AJ(I,K)*FAC
  102       CONTINUE
            S(I,J)=FAC
  103    CONTINUE
         DO 104 K=1,2
            AJ(I,K)=AJ(I,K)/DIAG
  104    CONTINUE
  105 CONTINUE
      DO 106 K=1,2
         AJ(NEQ,K)=AJ(NEQ,K)/S(NEQ,1)
  106 CONTINUE
C
C     BACK SUBSTITUTION
C
      DO 109 K=1,2
         DJ(NEQ,K)=AJ(NEQ,K)
         DO 108 I=1,NEQ1
            JROW=NEQ-I
            JEND=NEQ-JROW+1
            IF (JEND.GT.IBAND) JEND=IBAND
            RHS=AJ(JROW,K)
            DO 107 J=2,JEND
               JCOL=JROW+J-1
               RHS=RHS-S(JROW,J)*DJ(JCOL,K)
  107       CONTINUE
            DJ(JROW,K)=RHS
  108    CONTINUE
  109 CONTINUE
      RETURN
C
      END
```