**Comparison of LRFD and Allowable Stress Design Criteria for the Design of Multi-story Frames**

by

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(ABSTRACT)

Load and Resistance Factor Design is a set of specifications for the design of steel structures. It is based on a consistent probability of failure and the concept of limit states. The main difference between LRFD and the traditional Allowable Stress Design method is the use of load and resistance factors , which account for the variability of parameters affecting the design.

The purpose of this study is to compare both design methods. To do so, a computer program for the design of steel frames according to LRFD criteria is created. Three frame structures are designed using this program, then compared to the same structures designed according to the ASD procedures.

The results indicated that LRFD criteria yield a structure similar to that designed according to ASD, with a moderate saving in steel weight. Both methods were found to be similar in terms of complexity and effort to complete calculations. An advantage found in using LRFD was that engineers can develop a better understanding of the behavior of material and structures under the influence of different load conditions.

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# Chapter I

# Introduction

Probability theory plays an important role in safety and performance analysis, because it provides a more accurate engineering presentation of reality. Many of the variables affecting the performance of engineering systems are random in nature. Thus, uncertainties are unavoidable in the design of these systems. The engineering analysis of these systems should include methods and concepts for evaluating the effect and significance of uncertainty on the system performance and safety. Probability theory provides the mathematical basis for modeling uncertainty and for analyzing its effects in engineering design. Probability is a more rational approach to structural design because it provides a systematic and quantitative basis for dealing with uncertainty.

In the probabilistic approach to structural design, the applied loads and structural resistances are treated as random variables, and the design of the structure is based on the probability of failure which is obtain from a systematic analysis of the variabilities in the design parameters. Unlike the probabilistic approach, a deterministic approach to structural design compares stress from applied loads to allowable stresses given by a specification. Both applied loads and structural resistances are assumed to have one fixed value.

Many probability-based design formats have recently been proposed. Several European countries and Canada have incorporated or are considering the incorporation of probability-based codes. The Canadian Limit States Design Specification (Canadian Standard Association 1974) [8,11,12] is one example, of a probability-based code. Probability-based codes utilize the concept of limit states in conjunction with a second moment probability analysis to provide suitable levels of safety in design solutions. A limit state is a condition in which the structure or structural component can no longer satisfy its intended purpose.

The safety of a structure or structural component is measured by the probability of failure. This can be computed on the basis of probability distributions of the load and resistance variables and a functional relationship between these variables that define a given limit state. Since there is insufficient data to obtain the exact probability distributions of load and resistance variables, probability-based codes are based on a first-order second moment probability analysis. The first-order probability analysis uses the first two moments (mean and standard deviation) of the variables to compute the probability of failure.

After many years of research and testing, the American Institute and Steel Construction (AISC) has released a proposed Load and Resistance Factor Design (LRFD) Specification for structural steel buildings [18]. The proposed LRFD criteria is a set of structural steel design specifications that provides an alternative design procedure to the currently used AISC specifications. C.W. Pinkham and W.C. Hansell [17] describe LRFD as "a method of proportioning structural members and connections so that the strength and serviceability limit states exceed factored load combinations ". LRFD combines limit states with load and resistance factors to provide a set of checking procedures that are consistent with desired levels of reliability and performance. The concept of probability enters in the determination of suitable load and resistance factors used in the safety checking equations. The load and resistance factors in LRFD were obtained by calibration with existing Allowable Stress Design (ASD) criteria. Risk levels for structural elements designed in accordance with ASD specifications were computed using second order reliability analysis and target reliability indices were selected. The load and resistance factors were then computed using these target reliability levels. This make it possible for LRFD procedures to achieve similar levels of reliability as the existing ASD criteria.

The main purpose of LRFD is to provide a design procedure which results in uniform risk levels. In addition, the philosophy used in its development offers the opportunity to unify codes for all materials of construction.

## 1.1 Purpose and Scope

The primary purpose of this study is to compare designs of multi-story steel frames obtained using the proposed LRFD criteria with designs for the same frames obtained using the existing Allowable Stress Design (ASD) criteria. To accomplish this, three office buildings of two, five and ten story height were designed according to both methods. It is expected that the design solutions for structures using the new criteria should be similar to ASD design solutions. Since LRFD is a more rational and consistent methodology, however, the design solutions using the LRFD criteria may be more economical.

To accomplish this objective, a computer program adapted to the IBM Personal Computer, was developed. A program for designing structures based on this approach will serve as a useful tool for engineers interested in the design of steel buildings.

The program provides the user with the opportunity to evaluate several designs. These designs are compared to the designs of the same structures using the present ASD criteria. To satisfy the objective of this study, both methods are compared in terms of complexity of design procedure and in total steel weight. Since buildings of different heights were tested, the significance of height increase was also evaluated.

## 1.2 Organization

Chapter 2 presents a literature review of the LRFD method. The theory behind the procedure is discussed in detail, as well as the derivation of the load and resistance factors. The safety index which plays an important role in the reliability of a structure is defined. Some of the advantages of LRFD as well as differences with respect to ASD are also presented.

Chapter 3 presents the LRFD Design criteria for beams and columns for unbraced frame structures. The following topics are discussed: load combinations, design requirements for compression members, design requirements for flexural members, design for shear and interaction equations.

Chapter 4 presents a description of the procedure used to design multi-story buildings. The computer program for designing steel frames using the LRFD criteria is described.

Chapter 5 presents the designs of three multi-story frames used as examples for the comparison of the two design methods. A method to include P-Delta effects using linear analysis results is also presented. Results obtained from the LRFD and ASD are compared.

Finally, Chapter 6 contains a summary of the results and conclusions drawn from this study.

# Chapter II

# A Probability Based Code

## 2.1 General

Engineers have always recognized the importance of probability theory in safety and performance analysis. Many studies have been performed on the development of reliability-based design procedures [20,25,13,1]. Since the loads and structural resistances are random in nature, the performance of the structure affected should be treated in a statistical manner. Although, several European countries and Canada have adopted reliability-based codes, such codes are only recently being proposed for adoption in the United States. Some reasons for this are:

1. Lack of sufficient statistical data.

2. Probability-based design procedures can be somewhat complicated and standard building codes should remain simple in form.

3. Sufficient knowledge in reliability analysis was not available to calculate levels of safety (in a quantitative manner) for a particular structure [5].

In recent years, however, considerable research has been conducted on the development of reliability-based design codes and both AISC and ACI are considering the adoption of reliability-based codes.

Present codes for the design of steel structures do not account for the variability of different design parameters. Using the traditional approach, a beam subjected to a combination of dead and live loads is designed on the basis of an allowable stress. Both the structural resistance and loads are treated as deterministic quantities. The allowable stress, which is usually taken as a fraction of the yield strength of the member, is not related to any physically meaningful limit state [6]. To obtain the allowable stress, the yield strength of the material is modified by a safety factor. These factors of safety are obtained from engineering judgement and previous experience with similar structures. These factors have been modified throughout history to satisfy man's desire to build more economical structures [19]. Based primarily on previous successful designs, these factors are not necessarily the result of a rational analysis. Therefore, according to Freudenthal [8], "the resulting reliability is inconsistent, the designs are not optimal, and the ability to adapt to new situations is too slow".

Another disadvantage of the single safety factor approach is that a single number does not reflect the different variability of the different loads. For example, the same safety factor is applied to permanent and variable loads. According to Ellingwood [6], "because the dispersions in the latter are typically much higher, the reliability decreases significantly when the variable load contributes the major portion of the total load effect". Thus, the single safety factor approach is not a rational and reliable method for evaluating structural performance. For these reasons, the adoption of a reliability-based code promises to be a better approach.

In the probabilistic approach, the loads and resistances are treated as random variables and safety is defined by the probability of failure, which can be obtained from the probability distributions of the loads and resistances. In practice, however, information on the distributions is not easily available, so approximate methods are used. Of these, the most popular is the so-called second moment method, in which the reliability is obtained using the first two moments (the mean and standard deviation) of the variables. Safety is defined by the reliability index or the safety index, which is related to the probability of failure. The safety index provides a quantitative basis for evaluating the relative safety of several designs.

Many researchers have contributed to the development of these second moment methods. In Europe and Canada, Allen [ 1], developed second moment approaches. In North America, the first proposal was made by Cornell [ 5]. The studies by Galambos and Ravindra [8,9,20,25] have been the basis for a reliability-based code for steel construction.

An attempt to develop a reliability-based code for the design of steel structures began as a project sponsored by the American Iron and Steel Institute Committees of Structural Steel Producers and Steel Plate Producers. Load and Resistance Factor Design (LRFD) is the result of this project. It is a method of design for steel structures based on the concepts of limit states design and second moment reliability. Many individuals and groups helped in the development of the proposed method. A draft of the proposed AISC Load and Resistance Factor Design (LRFD) Specification for Structural Steel Building was first released for a one year period review and trial use began in September 1, 1983. In November 1984, the AISC Committee on Specification reviewed the results of the initial period and revised the document accordingly. A new version of the Proposed Specifications [18], used as the design reference in this study, was released in April 1, 1985. The current version (September 1986) is a legal document for professional use [15]. LRFD will be used as an alternative procedure to the ASD, which has been the basis for the Specifications for the Design, Fabrication and Erection of Structural Steel Buildings since its first edition in 1923. To help familiarize engineers with the new code, AISC has also published a guide to LRFD [23].

## 2.2 Organization of LRFD

The proposed LRFD specifications consists of several parts, as listed in Table 1. The arrangement of provisions differs from the ASD specifications in that the design guidelines are grouped according to structural components [9]. For example, all the criteria for columns and other compression members (such as effective length, buckling and design compressive strength) are in one section.

The section on loads and load factors is extensive, since they are treated differently than in ASD method. LRFD contains sections dedicated to strength and serviceability design considerations as well as fabrication, erection, and quality control. Additional appendices familiarize designers with the philosophy behind the specifications. This includes design aids and examples parallel to those in the previous ASD edition of the Manual. For example, Appendix F.1 contains a table which summarizes all parameters to be considered in the design of beams and girders [18]. Charts and maps are also provided as design aids. Only structural steels identified by the ASTM Specifications are approved for use with LRFD specifications.

## 2.3 Definition of LRFD

Load and Resistance Factor Design utilizes strength and serviceability limit states, combined with a first-order second moment probability analysis, to create a simple relationship that represents the interaction between the strength of the structure and the load applied on it. This relationship is expressed by the following inequality:

$$(\varphi R_n) \geq (\Sigma \gamma_i Q_i) \qquad [2.1]$$

**Table 1. Organization of LRFD**

A. General Provisions

B. Design Requirements

C. Frames and other Structures

D. Tension Members

E. Columns and other Compression Members

F. Beams and other Flexural Members

G. Plate Girders with Slender Webs

H. Members under Combined Stress, Tension and Combined Stress and Torsion

I. Composite Members

J. Connections, Joints and Fasteners

K. Strength Design Considerations

L. Serviceability Design Considerations

M. Fabrication, Erection and Quality Control

N. Evaluation of Existing Structures

Appendices

where,

$(\varphi R_n)$ is the factored nominal resistance (or ultimate strength) for a limit state.

$\varphi$ is the resistance factor, $\varphi < 1.0$ ; it accounts for uncertainties in the determination of the resistance of the material. It depends on the variability in material properties, geometric deviations and statistical differences between design models and experimental data.

$R_n$ is the nominal resistance of the structural element, determined in accordance with engineering principles.

$(\Sigma \gamma_i Q_i)$ required resistance, which is the summation of factored load effects for a critical load combination.

$\gamma_i$ is the load factor corresponding to $Q_i$. It accounts for uncertainties in individual loads.

$Q_i$ is the nominal load intensity.

$i$ is the type of load (that is dead load, live load, wind).

The left hand side of the equation corresponds to the structural capacity of the element provided by the material properties, while the right hand side represents the required resistance due to the critical combination of assumed loads. The resistance factor accounts for the possibility that the actual allowable strength is less than the nominal value of $R_n$. Likewise, load factors recognize the chance that load effects may vary from the nominal value of $Q_i$, as computed from assumed applied loads. These factors maintain a margin of reliability to account for unexpected deviations from nominal values.

For a satisfactory design, the factored nominal resistance should exceed the sum of all factored load effects on a particular element and limit state. A limit state is a condition in which a component ceases to satisfy its intended purpose. Limit states can be categorized into strength limit states and serviceability limit states. Strength limit states correspond to the limiting condition of failure, or collapse of the structure, when it reaches its maximum load carrying capacity. Serviceability limit states are the limiting conditions affecting the function of the structure during expected service. Examples of strength limit states for steel buildings include plastic strength in bending or tension, instability of local element or members or complete frame, and net section fracture. Serviceability limit states deal with excessive live load deflection or vibration, permanent deformation due to gross yielding or slip, and fatigue due to repeated service loads. Several limit states and load combinations can apply to a given element. For example, the design of a beam element should consider the following limit states: flexure about the major axis, shear, lateral torsional buckling, local flange and web buckling. The final design should be based on the critical limit state.

As an example of the application of Equation [2.1], consider a simple supported and laterally braced beam subjected to uniform dead and live loads. For the limit state of maximum bending moment of a beam, Equation [2.1] can be written as follows:

$$0.9ZF_y \geq 1.2\frac{w_{DL}l^2}{8} + 1.6\frac{w_{LL}l^2}{8} \qquad [2.2]$$

where,

| | |
|---|---|
| $\varphi$ | $= 0.9$ |
| $R_n$ | $= ZF_y$ |
| $\gamma_i$ | = 1.2 and 1.6, respectively |
| $Q_i$ | $= w_{DL}l^2/8$ and $w_{LL}l^2/8$, respectively |
| $Z$ | = plastic section modulus |
| $F_y$ | = specified minimum yield stress |
| $l$ | = length of member |

Given the loads, yield stress and length of the member, Equation [2.2] can be solved for the plastic section modulus, and the member can be sized to meet these requirement. Equation [2.1] can be written for each limit state and load combination applicable to the structural member to be designed.

To derive a workable design criteria, LRFD uses a second-moment reliability analysis. According to Ravindra and Galambos [8], "it is based on the assumptions that it is possible to estimate the second-moment statistical properties of the loads and the resistances, and that the load effects, Q, and the resistance, R, are independent random variables which can be represented by two distribution curves" as shown in Figure 1.

Failure of a structural member occurs when the structural resistance is less than the load effects due to applied loads,

$$R < Q \tag{2.3}$$

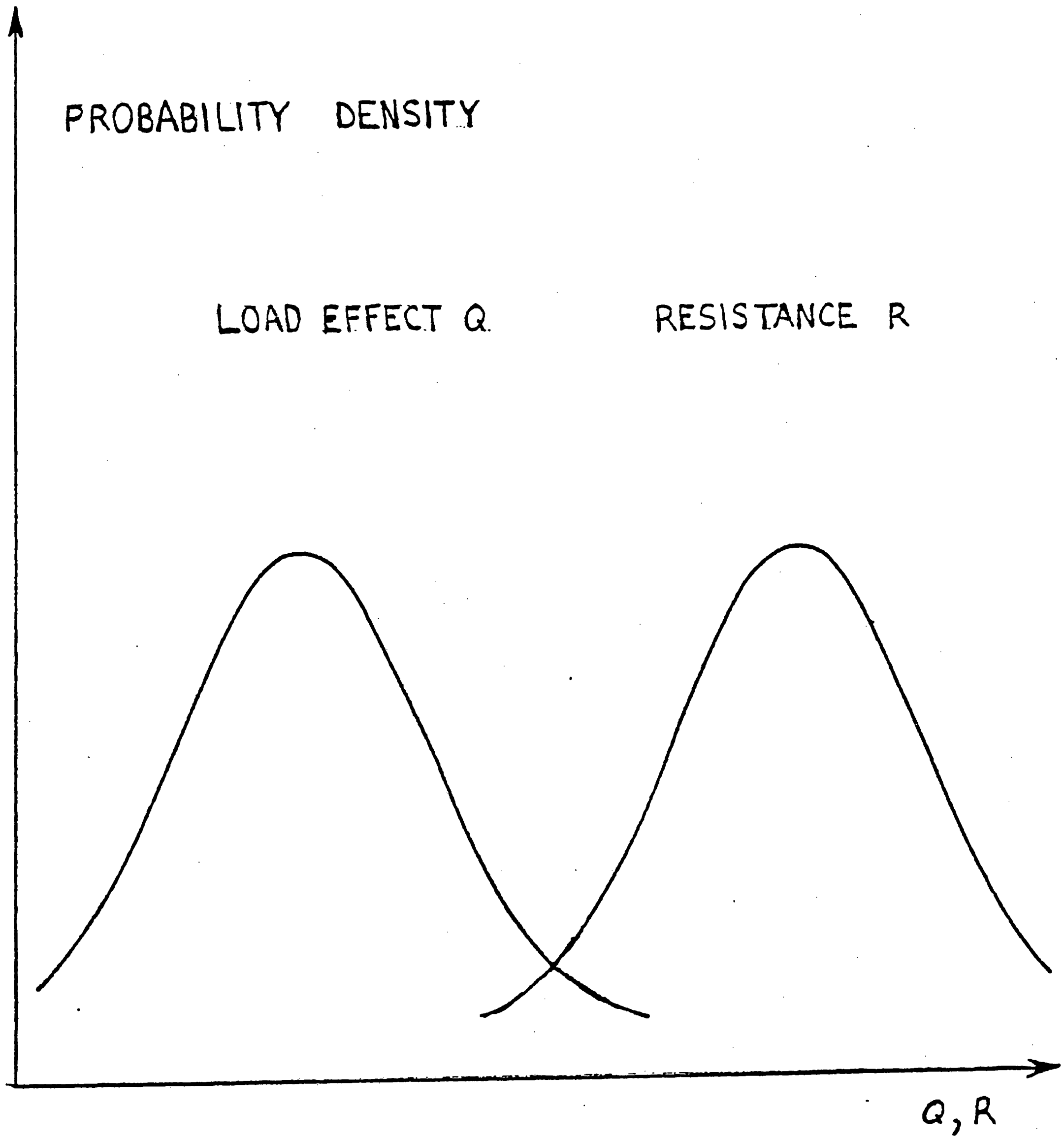
or,

$$\frac{R}{Q} < 1 \tag{2.4}$$

which can be written as

$$\ln\left(\frac{R}{Q}\right) < 0 \tag{2.5}$$

In this way, the frequency distributions of the two variables are combined into one, shown in Figure 2. The shaded area represents the probability of failure, or probability of reaching a limit state. Thus, the area to the right of the abscissa represents the probability of survival, or reliability of the structure. Reliability is defined by the "safety index", which is the ratio of the mean value of the combined frequency distribution, $\ln(R/Q)_m$, to its standard deviation, $\sigma_{\ln(R/Q)}$,




PROBABILITY OF FAILURE = $P(R \leq Q)$

**Figure 1. Probability distribution curves of the load and resistance variables**

$$\beta = \frac{\ln(\frac{R}{Q})_m}{\sigma_{\ln(R/Q)}} \qquad [2.6]$$

In graphical terms, the mean value is β standard deviations to the right of the safety margin. In reliability analysis, it is convenient to express the safety index, β , in terms of the mean value and the coefficient of variation. Assuming that the random variables R and Q are statistically independent, the standard deviation can be approximated by:

$$\begin{aligned} \sigma^2_{\ln(R/Q)} &\cong \left(\frac{\partial \ln(R/Q)}{\partial R}\right)^2_m \sigma^2_R + \left(\frac{\partial \ln(R/Q)}{\partial Q}\right)^2_m \sigma^2_Q \\ &= \frac{\sigma^2_R}{R^2_m} + \frac{\sigma^2_Q}{Q^2_m} = V^2_R + V^2_Q \end{aligned} \qquad [2.7]$$

Therefore,

$$\beta = \frac{\ln \frac{R_m}{Q_m}}{\sqrt{V^2_R + V^2_Q}} \qquad [2.8]$$

where $R_m$ and $Q_m$ are the means and $V_R$ and $V_Q$ are the coefficients of variation of the resistance, R, and load effect Q, respectively. Galambos and Ravindra [8] suggest that by keeping β constant for all structural elements, it is possible to obtain a uniform reliability.

The load and resistance factors in the LRFD criteria were obtained by calibration with the existing Allowable Stress Design Specifications [27,3,20]. Values of β were computed "by selecting a standard design situation and requiring that LRFD criterion produce the same design as the AISC Specification" [20]. β was plotted as a function of variables affecting the particular design situation. For example, Figure 3 shows the calibration for simple beams. The safety index varies from 2.6 to 3.4 [17]. Calibrations were also performed for simple columns, high strength bolts, and fillet welds[8, 7]. Studies of this kind were conducted at Washington Uni-

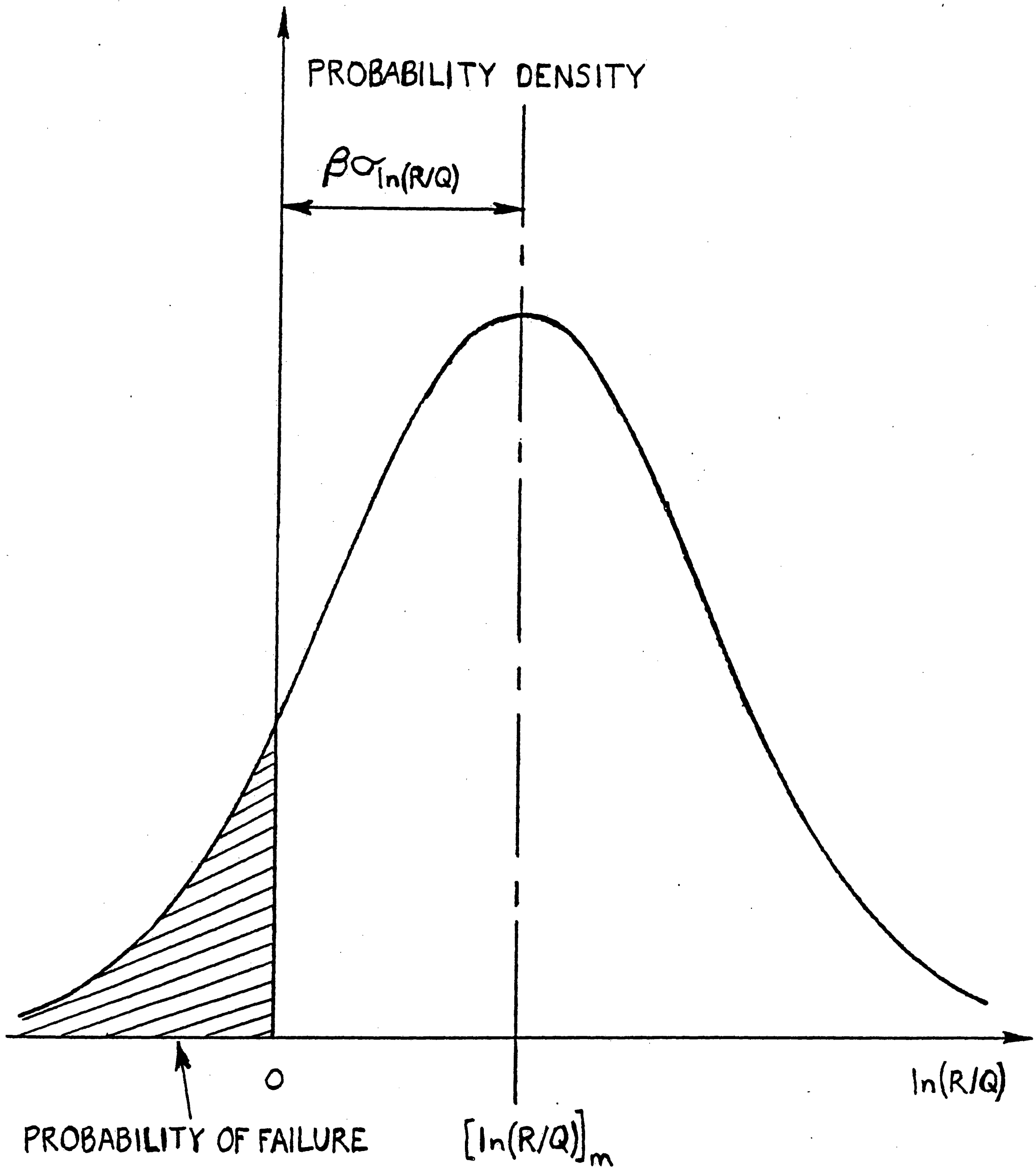



Figure 2. Definition of the safety index β

versity [17], and the conclusion was that $\beta = 3.0$ and $\beta = 4.5$ for members and connections, respectively, be used as the basis for developing load factors and resistance factors.

Separation is the final step in relating the safety index to the LRFD format. The resistance and load factors are obtained from the safety index as follows. Separating all resistance terms from Equation [2.8] and solving for mean resistance gives,

$$R_m = (Q)_m e^{\beta\sqrt{V_R^2 + V_Q^2}} \qquad [2.9]$$

the coefficient of variation term can be linearized as follows:

$$\sqrt{V_R^2 + V_Q^2} \cong \overline{\alpha}(V_R + V_Q) \qquad [2.10]$$

where $\overline{\alpha}$ is a separation constant. All resistance terms are moved to the left hand side of the equation,

$$R_m e^{(-\overline{\alpha}\beta V_R)} = Q_m e^{(\overline{\alpha}\beta V_Q)} \qquad [2.11]$$

The LRFD format is obtained after decomposing the load terms into different sources such as dead, live, and wind loads, where the resistance factors are:

$$\varphi = \frac{R_m}{R_n} e^{(-\overline{\alpha}\beta V_R)}$$

and the load factors have the form:

$$\gamma_i = 1 + \overline{\alpha}\beta V_i$$

where i corresponds to the type of load (that is dead, live, wind). Numerical studies by Ravindra and Galambos [17] found $\overline{\alpha} = 0.55$ to be an adequate value for the separation constant. The LRFD specifications contains tabulated numerical values of load and resistance factors for most common situations.

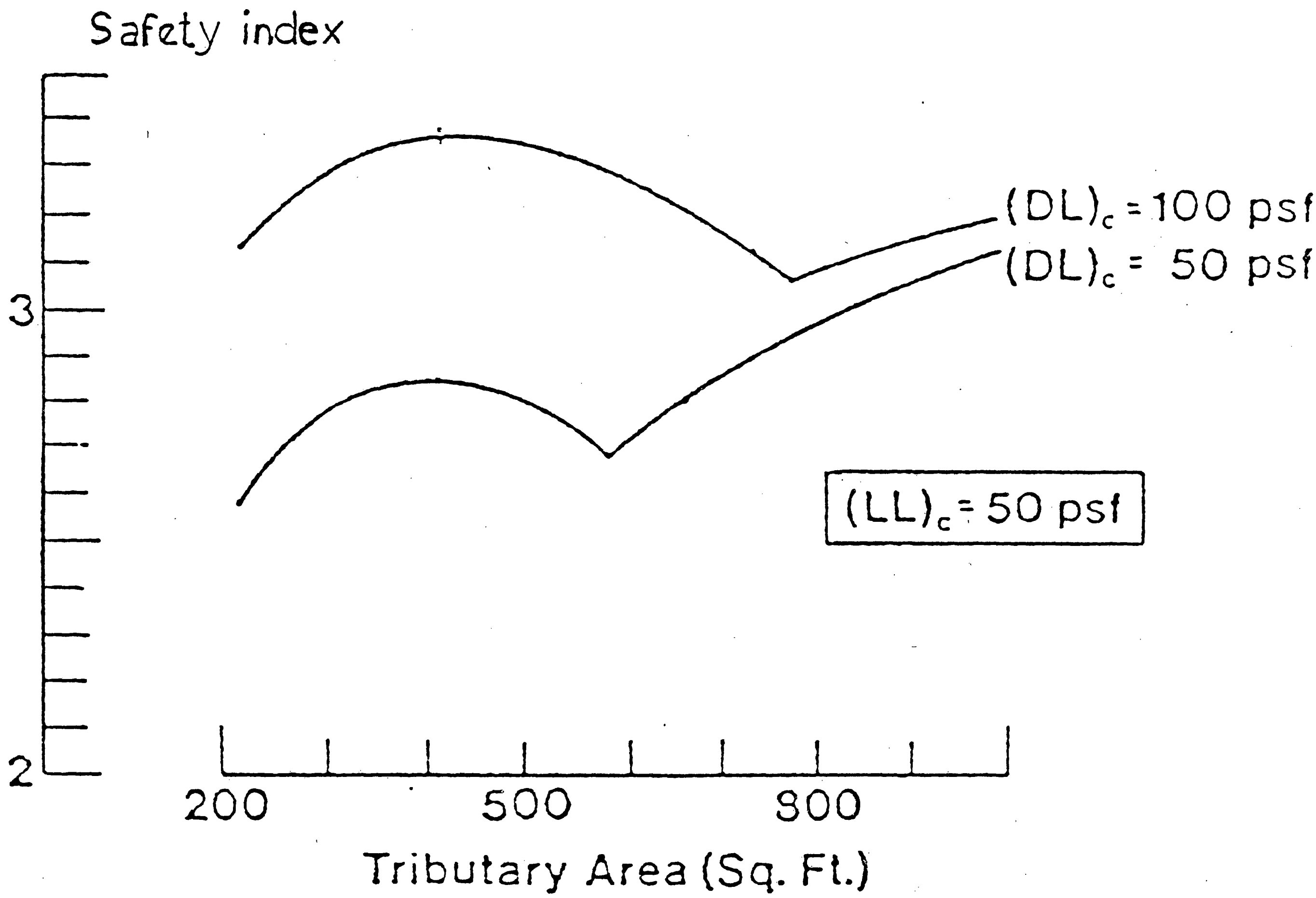



Figure 3. LRFD calibration for simple beams

## 2.4 Loads and Load Combinations

One of the major differences between LRFD and the Allowable Stress Design (ASD) is the approach used in the specification of loads. In the traditional ASD method, load intensity is usually specified as a deterministic value, say 20 psf for wind or 40 psf for a residential floor live load. But loads are not meaningful when described so simply. Water can increase in density by up to 4 percent after freezing, and other load sources may vary significantly. Hence, ASD does not prescribe loads in a realistic manner, because it does not account for the variability in load intensities. When more than one type of load acts on the structure, ASD combines the nominal values of these loads to compute the total design strength. This approach neglects the different variability that exists in these loads. Dead loads are always present in the structure and vary very little in comparison to live loads. Since ASD treates dead and live loads in the same manner, a uniform reliability is not possible.

LRFD uses a more rational approach towards the determination of load effect. Different loads have different variability, so different factors are used. As stated by Ellingwood [6], "loads that remain in place for long periods, such as dead load, tend to be less variable. In the other hand, loads applied for brief periods, such as wind load, tend to have larger variations". The greater the variability of a load type, the higher its corresponding load factor. Load combinations are based on one load being at its maximum value, while other loads are at their arbitraty-point-of-time values. It is unlikely that maximum lifetime loads from different sources will be present at the same time.

The load factors recommended by LRFD are the same found in ANSI A58.1-1982 [4]. They were developed using load statistics and second moment reliability analysis. Many years of research and experience were combined in obtaining suitable numerical values for these factors. As more data becomes available, the load factors will be refined and LRFD offers the means to do so.

## 2.5 Advantages of LRFD

Some of the differences between LRFD criteria and ASD specifications have been described in previous sections. Some of these differences, such as the treatment of loads and attainment of structural strength, are the concepts which make LRFD an improved design methodology for the construction of steel structures. The philosophy behind LRFD gives the new method several advantages over the traditional ASD criteria [14]. Some of these advantages are summarized in Table 2. LRFD emphasizes the importance of loads, by introducing different load factors and load combinations to the specification, making engineers more "load conscious". Since ASD applies the same margin of safety to all load sources, engineers are not aware of the existing difference of variability of these loads. Because loads are treated in a more realistic manner, a more uniform reliability is possible. One of the advantages of keeping a uniform reliability in design procedures is that structures designed on these basis may be more economical.

LRFD incorporates the principles of limit states to its provisions and provides the design engineer with an improved understanding and feel for loads and the structure resisting them. Limit state design provides wider applicability of criteria to non-routine problems, by helping engineers make appropriate adjustments in structures when they are unusual. It also facilitates designing structures that are composite, such as steel and concrete. As new or improved information on the strength of steel members becomes available, LRFD will simplify their incorporation to the Specifications by indicating what data is required and how it can be introduced without losing a uniform level of structural safety [14].

Since the same reliability analysis used in LRFD can be applied to any other discipline, it is possible to unify codes in structural design by maintaining a common safety index. In the future, it will simplify comparisons between alternate designs of different materials. Similar studies of probabilistic limit state criteria are being conducted for concrete, masonry, timber construction. In fact, the American National Standards Institute (ANSI) Committee A58, "Building Code Requirement for Minimum Design Loads in Buildings and other Structures", was revised in 1982 by a subcommittee which made use of the probabilistic limit states design approach to develop a common load criteria applicable to all construction materials [14].

In conclusion, LRFD provides the engineer with a more realistic view of the parameters affecting the design of steel structures. Because of all its benefits, the adoption of LRFD offers the opportunity for future progress in the art of structural design.

**Table 2. Summary of some of the advantages of LRFD**

1. Realistic treatment of loads and load combinations
2. Uniform reliability resulting in economical designs
3. Incorporation of the principles of limit states in the Specifications resulting in a better understanding of structure behavior
4. Wider applicability to non-routine problems
5. Easy incorporation of new information to the Specifications
6. Ability to unify codes in structural design
7. Simplification of comparisons between designs of different materials

# Chapter III

# LRFD Design Criteria for Beams, Columns and Beam-Columns

Multistory buildings are common examples of frame structures. They can be defined as a combination of columns and beams joined together by fixed or pin connections. This study assumes that the members are connected by fixed (or moment resisting) connections.

Steel members in multistory buildings and other structures are subjected to loads from several sources. This study considers only dead, live and wind loads. All columns are assumed to be in compression because dead load is predominant over the other loads and produces compression in these members. Since the frames studied have fixed joints, both buckling due to axial loads and, combined axial load and end moment forces are taken into consideration in the design of columns. The horizontal end forces on the columns are usually very small so they can be neglected for design purposes. Beams are designed for shear and bending moment effects. In this case, axial forces are negligible so they are not taken into consideration during the design process. Both beams and columns are designed using the LRFD specifications.

## 3.1 Load Combinations

LRFD recommends the following combinations of factored loads:

$$1.4D \tag{3.1}$$

$$1.2D + 1.6L + 0.5(L_R \text{ or } S \text{ or } R) \tag{3.2}$$

$$1.2D + 1.6(L_R \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \tag{3.3}$$

$$1.2D + 1.3W + 0.5L + 0.5(L_R \text{ or } S \text{ or } R) \tag{3.4}$$

$$1.2D + 1.5E + (0.5L \text{ or } 0.2S) \tag{3.5}$$

$$0.9D - (1.3W \text{ or } 1.5E) \tag{3.6}$$

where,

$D$ = dead load due to the weight of the structural elements and the permanent fixtures on the structure.

$L$ = live load due to occupancy and moveable equipment.

$L_R$ = roof live load.

$W$ = wind load.

$S$ = snow load.

$E$ = earthquake load.

$R$ = nominal load due to initial rainwater or ice exclusive of ponding contribution.

Since this study only considers dead, live and wind loads, the above equations are reduced to:

$$1.4D \tag{3.7}$$

$$1.2D + 1.6L \tag{3.8}$$

$$1.2D + 0.5L \tag{3.9}$$

$$1.2D + 0.8W \tag{3.10}$$

$$1.2D + 1.3W + 0.5L \tag{3.11}$$

$$1.2D + 0.5L \tag{3.12}$$

$$0.9D - 1.3W \tag{3.13}$$

Equations [3.9] and [3.12] are not included in the set of load combinations that is used in the computer program, because they lead to smaller numerical values than equation [3.8]. The set of factored load combination equations used in the computer program developed from this study to determine the critical required strength of a structure is thus reduced to five.

## 3.2 Design Requirements for Compression Members.

For compression members whose elements have width-thickness ratios less than $\lambda_R$ defined in Sect. B5.1 of the specifications [15], LRFD recommends that the required compressive strength shall be less than $\varphi_c P_n$

$$\begin{aligned} \varphi_c &= 0.85 \\ P_n &= A_g F_{cr} \end{aligned} \tag{3.14}$$

where $A_g$ = gross area of the member, $in^2$

for $\lambda_c \leq 1.5$

$$F_{cr} = \left[0.658^{\lambda_c^2}\right] F_y \tag{3.15}$$

for $\lambda_c > 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \qquad [3.16]$$

where,

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} \qquad [3.17]$$

$F_y$ = specified minimum yield stress, ksi

E = modulus of elasticity, ksi

K = effective length factor

L = unbraced length of member, in

r = governing radius of gyration about plane of buckling, in

## 3.3 Design Requirement for Flexural Members

LRFD requires that the critical factored bending moment shall be less than the flexural design strength, $\varphi_b M_n$

where,

$\varphi_b$ = 0.9

$M_n$ = nominal strength

To meet local buckling specifications, steel sections are classified as: compact, non-compact and slender element sections. Compact sections are defined as having their flanges continuously connected to the web or webs and the width-thickness ratios of their compression elements must not exceed the limiting width-thickness ratio $\lambda_p$ from Table B5.1 [15]. A noncompact section is one in which the width-thickness ratio of one or more compression elements exceeds $\lambda_p$ but does not exceed $\lambda_r$. Finally, a slender element is one whose width-thickness ratio exceeds $\lambda_r$, also found in Table B5.1 [15]. The nominal strength is determined as follows:

1. For compact section members with $L_b \leq L_p$

$$M_n = M_p = ZF_y \qquad [3.18]$$

where,

$$L_p = \frac{300r_y}{\sqrt{F_y}}$$

$L_b$ = laterally unbraced length, in

$Z$ = plastic modulus, $in^3$

$F_y$ = yield stress, ksi

2. For compact section members with $L_p < L_b \leq L_r$

$$M_n = C_b\left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \qquad [3.19]$$

where,

$C_b$ = $1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 2.3$; $M_1$ is the smaller and $M_2$ the larger end moment in the unbraced segment of the beam; $M_1/M_2$ is positive when the moments cause reverse curvature and negative when bent in single curvature.

$C_b$ = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$$L_r = \frac{r_y X_1}{F_y - F_r}\sqrt{1 + \sqrt{1 + X_2(F_y - F_r)^2}} \qquad [3.20]$$

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \qquad [3.21]$$

$$X_2 = \frac{4C_w}{I_y}\left[\frac{S_x}{GJ}\right]^2 \qquad [3.22]$$

$$M_r = S_x(F_y - F_r) \qquad [3.23]$$

$S_x$ = Section modulus about major axis, in$^3$

$E$ = Modulus of elasticity of steel (29000 ksi)

$G$ = Shear modulus (11200 ksi)

$J$ = Torsional constant, in$^4$

$A$ = Cross-sectional area, in$^2$

$C_w$ = Warping constant, in$^6$

$F_r$ = Compressive residual stress in flange; 10 ksi for rolled shapes, 16.5 ksi for welded shapes.

3. For compact section members with $L_b > L_r$

$$M_n = M_{cr} \le C_b M_r \qquad [3.24]$$

where $M_{cr}$ is the critical moment

$$M_{cr} = C_b \frac{\pi}{L_b}\sqrt{EI_yGJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \qquad [3.25]$$

$$= \frac{C_b S_x X_1 \sqrt{2}}{L_b/r_r}\sqrt{1 + \frac{X_1^2 X_2}{2(L_b/r_y)^2}}$$

## 3.4 Design for Shear

Most of the shear applied to the member shall be taken by the web. The critical factored shear shall be less than the design shear strength, $\varphi_v V_n$

where,

$\varphi_v = 0.9$

$V_n$ = nominal shear strength

The nominal shear strength is determined as follows:

1. For $h/t_w \leq 187\sqrt{k/F_y}$

$$V_n = 0.6F_y A_w \qquad [3.26]$$

2. For $187\sqrt{k/F_y} < \frac{h}{t_w} \leq 234\sqrt{k/F_y}$

$$V_n = 0.6F_y A_w \frac{187\sqrt{k/F_y}}{h/t_w} \qquad [3.27]$$

3. For $h/t_w > 234\sqrt{k/F_y}$

$$V_n = A_w \frac{26400k}{(h/t_w)^2} \qquad [3.28]$$

where,

$h$ = web depth, in

$t_w$ = web thickness, in

$A_w$ = area of web, $ht_w$, in$^2$

$k$ = web plate buckling coefficient given by

$$k = 5 + \frac{5}{(a/h)} \qquad [3.29]$$

where a is the clear distance between transverse stiffeners, in; k is taken as 5 for rolled shapes.

## 3.5 Steel Members under Combined Stresses

In most cases steel columns are subjected to combined effect of axial forces and bending moments. The design of such members are based on interaction equations. For columns subjected to axial and bending forces, the following interaction equations apply:

for $\dfrac{P_u}{\varphi_c P_n} \geq 0.2$

$$\frac{P_u}{\varphi_c P_n} + \frac{8}{9}\left(\frac{M_u}{\varphi_b M_n}\right) \leq 1.0 \qquad [3.30]$$

for $\dfrac{P_u}{\varphi_c P_n} < 0.2$

$$\frac{P_u}{2\varphi_c P_n} + \frac{M_u}{\varphi_b M_n} \leq 1.0 \qquad [3.31]$$

where,

$P_u$ = required compressive strength, kips.

$P_n$ = nominal compressive strength determined by Section 3.2, kips.

$M_u$ = required flexural strength determined according to the paragraphs below, kip-in.

$M_n$ = nominal flexural strength determined according to Section 3.3, kip-in.

$\varphi_c$ = resistance factor for compression, $\varphi_c = 0.85$

$\varphi_b$ = resistance factor for flexure, $\varphi_b = 0.9$

**Determination of Mu:** Frames that are unsymmetrical or subjected to unsymmetrical loading have the tendency to sidesway, due to the unequal moment distribution. Therefore, additional moments are created by axial compression forces acting through the lateral displacements of the columns. This phenomenon is called the P-Δ effect. To account for these second order effects, LRFD suggests two methods to determine the required flexural strength, $M_u$, to be used in the interaction equations. The choice of either one is left to the engineer's judgement.

a. In structures designed on the basis of elastic analysis, $M_u$ may be determined from a second order elastic analysis, in which equilibrium is formulated on the deformed structure. This analysis shall be made using factored loads.

b. When the structure is designed on the basis of a first order elastic linear analysis, the following procedure can be used instead of the method described above:

$$M_u = B_1 M_{nt} + B_2 M_{lt} \qquad [3.32]$$

where,

$M_{nt}$ = required flexural strength in the member assuming there is no lateral translation of the frame, kip-in.

$M_{lt}$ = required flexural strength in member as a result of lateral translation of the frame only, kip-in.

The factor $B_1$ is determined as follows :

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_e}\right)} \geq 1.0 \qquad [3.33]$$

where,

$P_e = A_g F_y / \lambda_c^2$, where $\lambda_c$ is defined by Equation [3.17] with $k \leq 1.0$ in the plane of bending.

$C_m$ = coefficient whose value shall be taken as defined in Section H2.a [15].

The factor $B_2$ is determined from:

$$B_2 = \frac{1}{1 - \Sigma P_u \left( \frac{\Delta_{oh}}{\Sigma HL} \right)} \qquad [3.34.1]$$

or alternatively,

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma P_e}} \qquad [3.34.2]$$

where,

$\Sigma P_u$ = required axial load strength of all columns in a story, kips.

$\Delta_{oh}$ = translation deflection of the story under consideration, in.

$\Sigma H$ = sum of all story horizontal forces producing $\Delta_{oh}$, in.

L = story height, in.

$P_e$ = $A_g F_y / \lambda_c^2$, where $\lambda_c$ is the slenderness parameter defined by Equation [3.17], based on the actual $kL/r$, and shall not be less than unity, kips.

Note that $B_1$ accounts for the secondary P-Δ effect in all frames and $B_2$ covers the P-Δ effect in unbraced frames.

*

# Chapter IV

# Design of Multi-story Frames

## 4.1 General

One of the purposes of this study was to develop a computer program to help in the design of steel frames using the LRFD Specifications. This program designs beams, columns and beam-columns. The design of multi-story buildings can be very tedious, if the number of members in the frame is large. With the aid of a microcomputer, this task can be performed more efficiently.

The program developed for this study is written in the MICROSOFT interpreter version of the BASIC language for the IBM family of personal computers. Because it is the most widely used language for microcomputers and is simple to use [22], BASIC was chosen as the programming language. It is a more friendly environment to work in and it makes the entire process of developing and testing the program much easier.

The program was developed using the structured approach. The program is divided into a number of modules, each having its own function and being independent of the others. A top-down approach is used to create these modules. Each subroutine (module) is added one at a time at the end of the program, and then tested for correctness.

## *4.2 Structure of Program*

The flow diagram of the program for the analysis and design of frame structures is shown in Figure 4. It consists of three modules: FRAME, DESBEAM and DESCOLM. The module FRAME is a plane frame analysis program which computes member end forces and member end moments by the stiffness method. The module DESBEAM designs beams in accordance to the LRFD criteria, and the module DESCOLM designs both columns and beam-columns in accordance to the LRFD Specifications. The three modules are executed separately, but are linked together using the BASIC CHAIN command, described further in the following section. By chaining the three programs it is possible to completely automate the analysis and design processes.

The input to the program is from a data file containing information on the preliminary design. This information includes the structure configuration, member properties and loading. FRAME is the first module to be executed. It performs a matrix structural analysis to find member end forces and moments. After the structure is analysed, FRAME creates two data files, BEAMF.DAT and COLUMNF.DAT, containing member end forces and moments of beams and beam-columns, respectively. The data file BEAMF.DAT is used by the beam design module DESBEAM, and the data file COLUMNF.DAT is used by the column design module DESCOLM. After the creation of these data files, the first module FRAME is deleted from memory and replaced by the module DESBEAM. This second module, DESBEAM, uses the information stored in data file BEAMF.DAT to design all of the beams of the frame using the LRFD Specifications. It is then deleted from memory and replaced by the third module, DESCOLM, which uses the data file COLUMNF.DAT to design all of the columns of the frame according to LRFD criteria. This constitute the end of one design cycle.

After the end of a design cycle, the user can modify the original data file and change the member properties using the new member sizes selected by the beam and column design modules. Then, the program is executed again to obtain a new design solution. This procedure is repeated until the member sizes are equal or similar to those from the previous design cycle. Time for completion and the number of trials required to converge to a given design will vary depending to the number of members in the frame and on the quality of the preliminary design.

## 4.3 *Special Features of the Program*

The three modules can be executed independently, but they are linked together by the BASIC CHAIN command to avoid having to transfer data from one module to another. The BASIC CHAIN command provides the mechanism for transferring control from one program to another. When transferring control to another program it is important to preserve the values of some of the program variables so that these can be used by the new program. There are several ways of transferring variables from one program to another; the two most popular methods being:

1. To use a COMMON statement to define which variables (array or scalar) will remain active in memory and will be transferred from the previous program to the new program.
2. To create a data file to store the values of these variables, and require the new program to read this data during execution.

The latter option was used in the design programs, since it allows for greater storage efficiency.

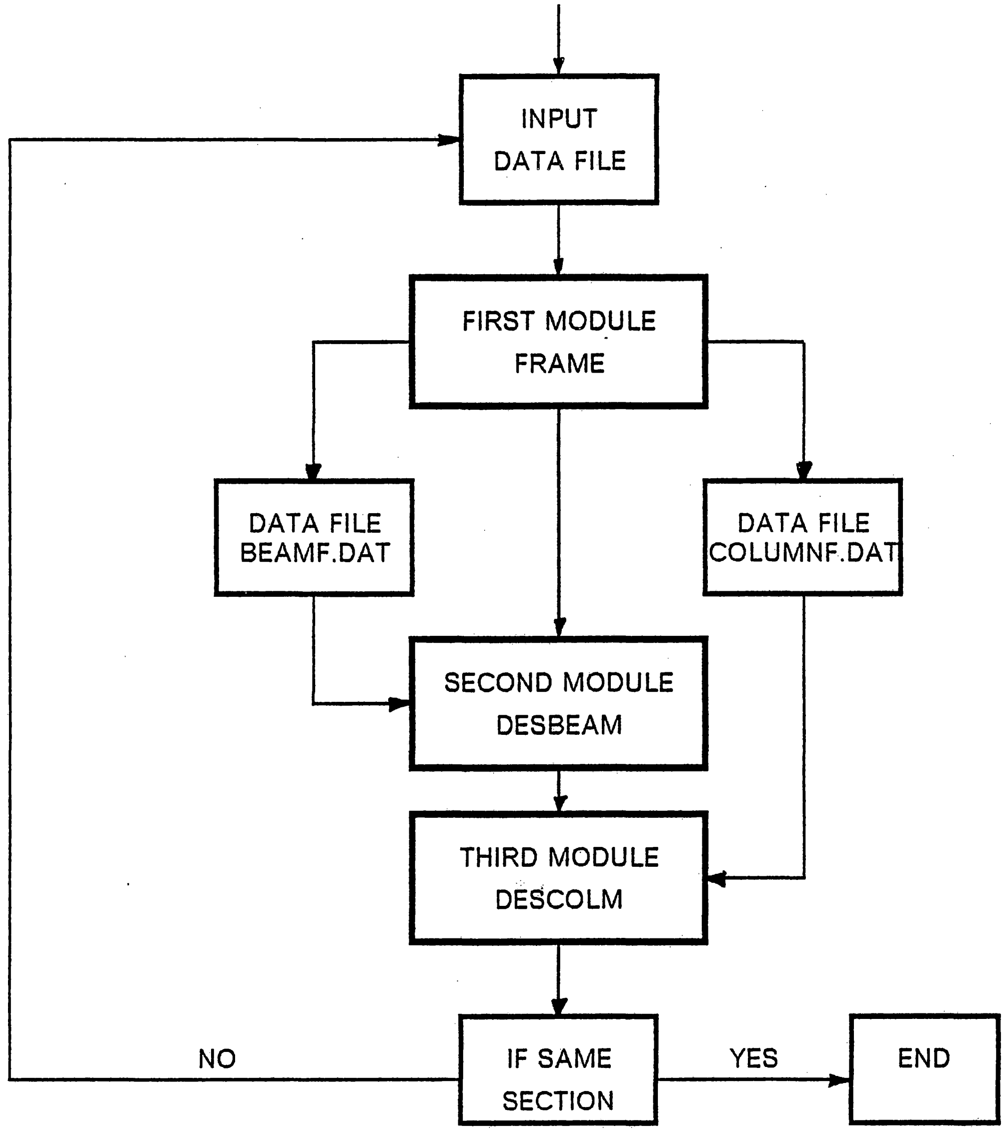



**Figure 4. Diagram of the multi-story frame design procedure**

Another feature of the program is the use of data files for input and output of data. Although, data can be entered interactively, this approach requires the user to type in the information each time the program is executed. Since the amount of information required to analyse and design a multi-story frame is large, this approach would be too time-consuming and can lead to errors. Therefore, data files are used to provide input to the program.

## 4.4 Data Files

Five sequential data files are used during the execution of the program. The user needs to create only one data file, the input file. The other four are either automatically created by the program or already exist on disk. The input data file contains the following information separated by commas (units are inches, kips and radians):

1. Number of elements, number of joints and number of load conditions.
2. Member incidences.
3. Joint number and joint direction for each joint constraint. Type 0,0 after the last set of data, or if there are no joint constraints.
4. Joint coordinates (x,y) for each member.
5. Member properties: area, moment of inertia, modulus of elasticity, unbraced length, effective length factor about the x-axis and effective length factor about the y-axis, for each member.
6. For each load condition: joint number, joint direction and factored forces. Type 0,0,0 at the end of the last set of data, or if there are no joint loads.
7. For each load condition: member number, member action type, magnitude of action (factored load), and location of application. Type 0,0,0,0 at the end of the last set of data, or if there are no member action.

The module FRAME creates two files required for the execution of modules DESBEAM and DESCOLM. The data file BEAMF.DAT for the module DESBEAM, contains the following information:

1. Number of elements.
2. For each beam: member number, length, unbraced length.
3. For each load condition: member number, load condition number, member action type, magnitude of action, location of application.
4. For each load condition: member number, axial, shear and moment end forces.

The data file COLUMNF.DAT, for module DESCOLM, contains the following information:

1. Number of elements.
2. For each beam-column: member number, length, unbraced length, slender ratio $k_x$, and slender ratio $k_y$.
3. For each load condition: axial, shear and moment end forces.

Two other files are also required for the execution of the design modules. These data files contain section properties of available W-sections, as listed in the AISC Manual [2], for use in the design of beams and columns. The data file COLUMNF.DAT contains section properties for the design of column, while the data file BEAMF.DAT contains section properties for the design of beams.

## 4.5 Frame Analysis Module

The first module, FRAME, performs a matrix structural analysis and computes factored load effects, including axial, shear, and moment end forces, for each member in the structure. This module is the microcomputer version of a matrix analysis program [16] with a few modifications. The program was modified so that data could be entered from data files. A diagram of the BASIC version of this module is shown in Figure 5. Another modification that was made to the original program was the addition of a last subroutine FILES. This subroutine determines which members are beams and which are beam-columns. It then stores the end forces for the beams and beam-columns, obtained from the analysis, in the data files BEAMF.DAT and COLUMNF.DAT, respectively.

## *4.6 Design of Beams*

The second module, called DESBEAM, contains seven subroutines. Figure 6 shows these subroutines and their sequence of execution. The beam design module performs the following functions: it reads the data file BEAMF.DAT and organizes the information so that it can be processed by the program. Then, a subroutine (ANALYSISB) computes maximum factored shears and moments for each beam in the frame structure and for each load combination. Shear and bending moments at twenty sections along each beam are computed for each load combination using factored loads and factored load effects. Then, the maximum required shear and moment is computed for each beam. These values, as well as the load combination number, are printed on the screen.

Having completed the analysis, a separate subroutine (DESIGNB) designs each beam by the LRFD Specifications. It computes $C_b$ coefficients and selects a section from a data file containing available beam W-section listed in the AISC Manual [2]. The limit states of flexure, shear, lateral torsional buckling, local flange buckling, and local web buckling are checked. The computed nominal moment and shear are compared to the factored bending moment and factored shear, respectively. If the section is satisfactory, the program compares its weight per linear foot with the current minimum weight. If it is less than the current minimum weight section, the program saves the weight of the section as the new minimum weight section. Otherwise, the section is discarded. This process is repeated until all sections listed in the

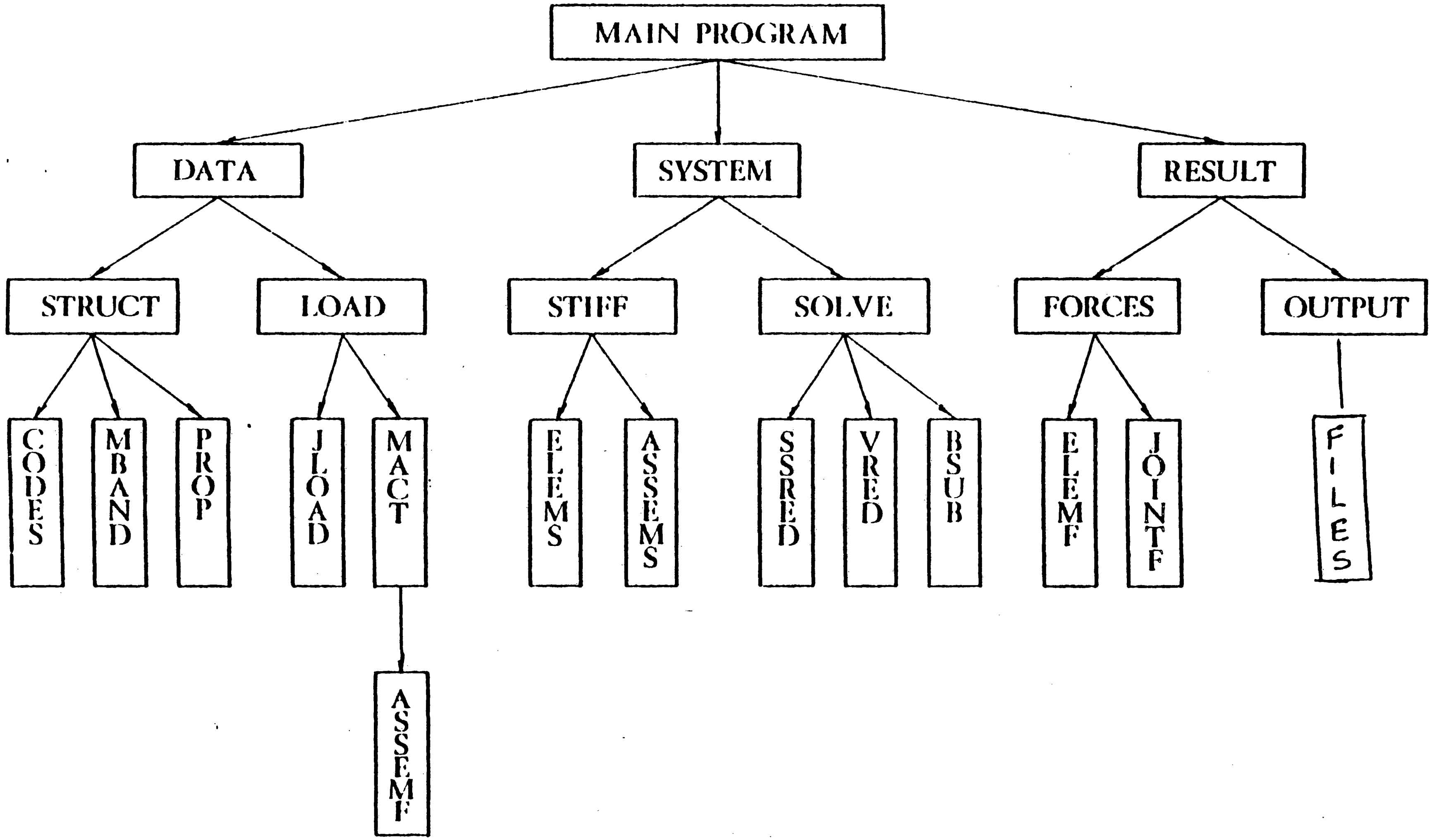



Figure 5. Diagram of the BASIC Version of frame analysis module

data file are checked. The resulting section is the lightest for the given loading. This procedure is repeated for all the beams in the frame.

The last subroutine prints the results on the screen for each beam. The output includes member number, member size, required factored moment, required factored shear, nominal shear, and moment resistances.

## 4.7 Design of Columns

A diagram of the BASIC version of the third module is shown in Figure 7. It contains seven subroutines and performs the following tasks: It reads information from a data file COLUMNF.DAT. After entering the factored axial and moment end forces for the first load combination, it computes the required compressive strength and flexural strength. These values are printed on the screen for each beam-column. Then, a separate subroutine (DESIGNC) designs each beam-column using the LRFD criteria. It computes $C_b$ coefficients and selects a section from a data file containing available beam-column W-sections as listed in the AISC Manual [2]. It checks if the section satisfies LRFD criteria for members under combined stresses, and determines the nominal compressive strength, the nominal flexural strength, and the appropiate interaction value. If the interaction value is less than unity, the section is acceptable. Then, it compares the weight per linear foot of the section to the current minimum weight. If the weight of the section is less than the current minimum weight section, it is saved as the new minimum weight. Otherwise, it is discarded. This process is repeated until all sections in the data file are checked, thus giving the lightest column W-section that meets LRFD criteria for the given load combination. The last subroutine prints the results on the screen and on the printer, including member number, member size, interaction coefficient, required compressive strength, required flexural strength, compressive resistance, and flexural resistance.

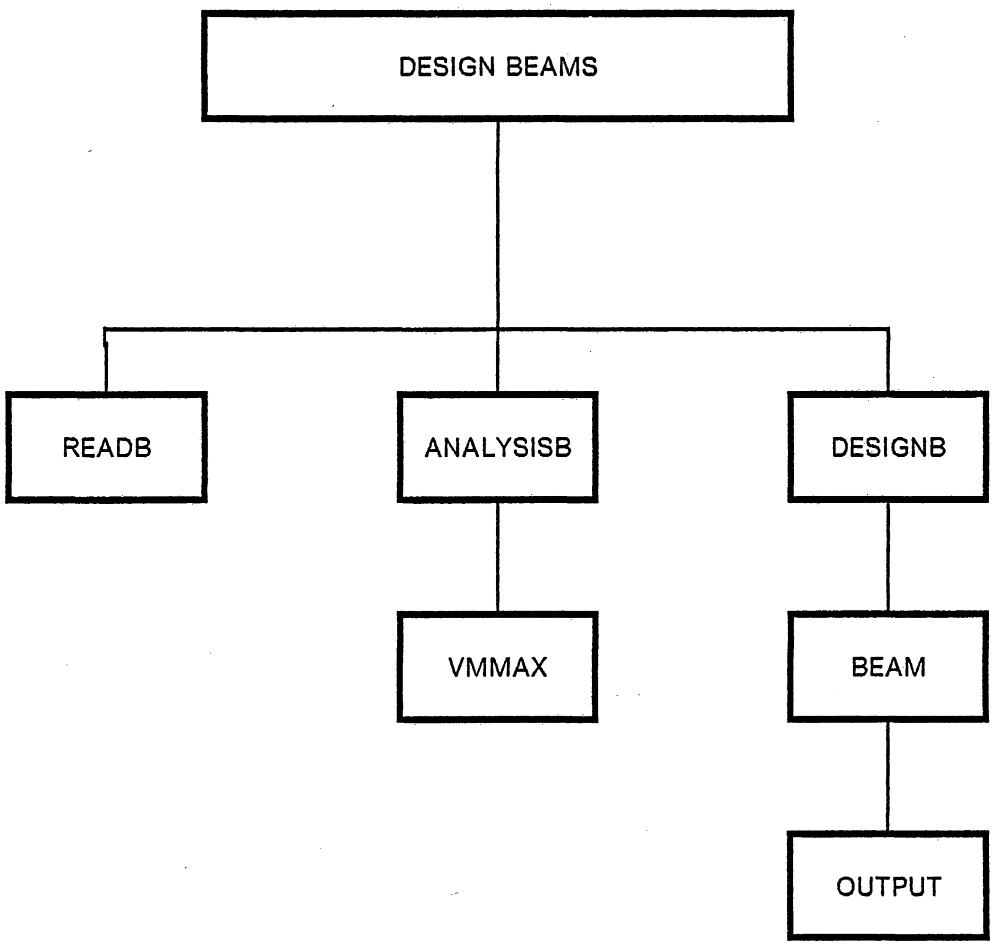



**Figure 6. Diagram of BASIC Version of beam design module**

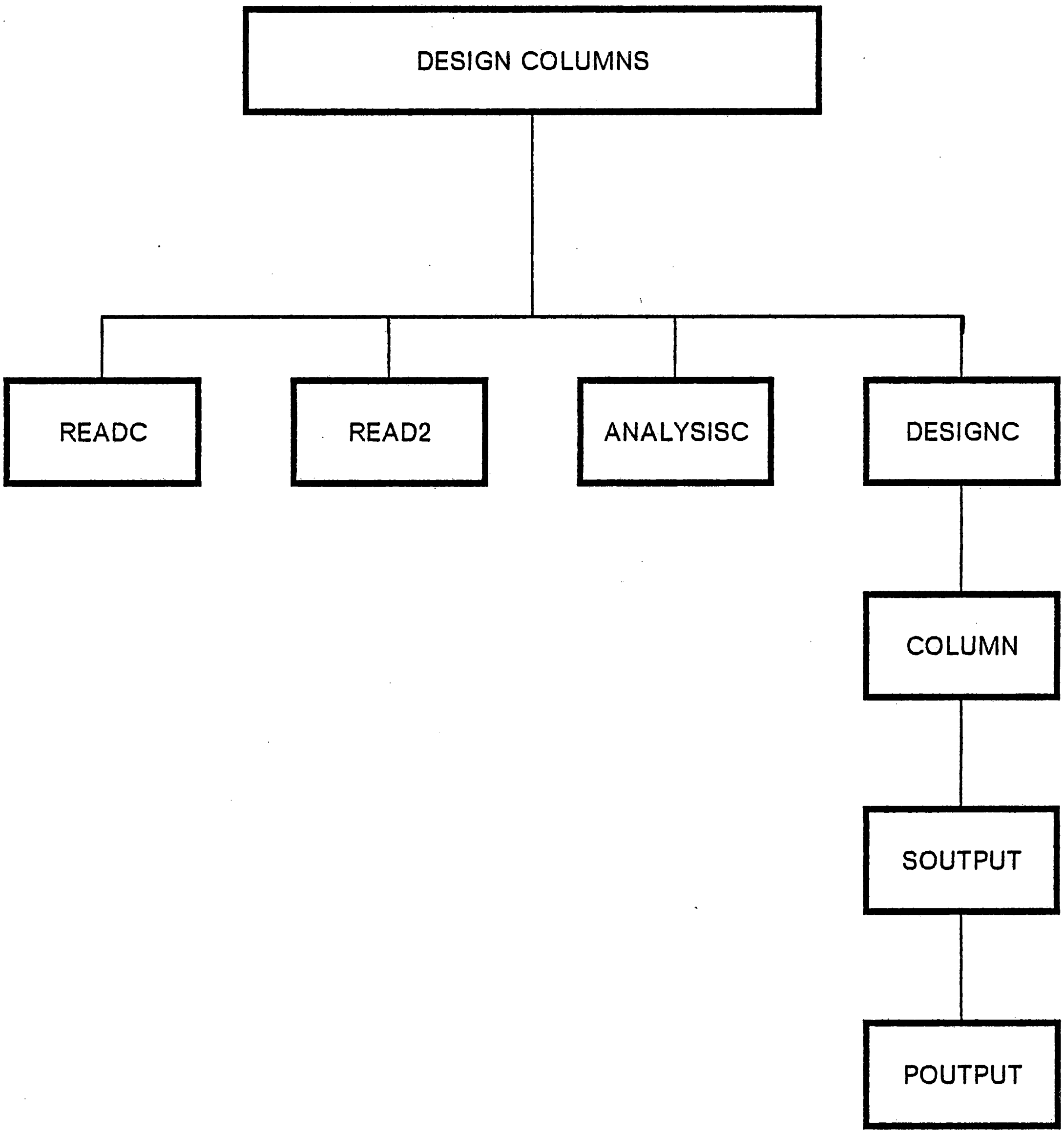



**Figure 7. Diagram of BASIC Version of the column design module**

The factored axial and moment end forces for the next load combination are read and the procedure to find the lightest section is repeated. Hence, the execution of the entired module results in five different design solutions for each beam-column, one for each loading combination; the user, then, selects the best section from these design solutions.

# Chapter V

# Comparison of LRFD and Allowable Stress Design

The main purpose of this study was to compare the LRFD procedure and the Allowable Stresss Design method. To accomplish this, multi-story frames of different heights were designed using the two specifications. In this chapter, designs obtained using the two procedures are presented, and the results are compared.

Three steel frames, a two-story, a five-story, and a ten-story frame were designed by both methods. The plan and elevation of the three frames are shown in Figure 8, 9, and 10, respectively. The frames are subjected to dead, live, and wind loads. These design loads are the same as those used in the design example of a 25-story building in Reference [21]: wind load = 20 psf, live load = 50 psf, and dead load = 75 psf. LRFD requires the use of load factors and several combinations of these factored loads.

On the other hand, when using ASD, the Steel Design Manual [24] recommends that the allowable stress can be increased by 33% if wind loads act in combination with dead and live loads, provided that the required section computed on this basis is not less than that required for the design of dead and live loads computed without the 1/3 stress increase. This factor accounts for the probability of all three loads acting simultaneously. With this in mind, two load combinations are considered: one, where all loads are multiplied by a factor of 0.75 to account for that 1/3 increase in stress, and two, where only dead and live loads are applied to the structures.

Member selection is based on the 1977 table of dimensions and properties published by AISC [2]. Only rolled W-sections are considered in the design. The design of columns is closely related to the beam sizes. Because there is no explicit relationship available to determine the necessary beam sizes and column sizes for which the design solution is optimum, the attainment of an optimum design depends on the skill and experience of the designer. Since the objective of the study was to compare two design procedures, rather than to obtain an optimum design solution, the following sections were considered: for columns, W14 sections provide a large range of available sections; for beams, only W18 sections (and W21 for the five-story building) are considered.

Both LRFD and ASD design solutions are based on member end forces obtained from a matrix structural analysis program [16]. LRFD, however, requires that a second-order analysis be performed using factored loads or in lieu of such an analysis, an approximate method of analysis can be used.

In an attempt to obtain realistic design solutions the following conditions were imposed: the columns within a row are kept identical in depth; beams and columns are symmetrical within each story; connections are assumed to be rigid, including fixed columns bases; only steel of yield grade 36 is considered; center to center spans and heights are used for the analysis; all members at any joint meet at a point; the effects of differential length changes in all members are negligible.

## 5.1 Approximate method to calculate P-Delta effects

First-order analysis does not account for the changes in internal moments due to axial loads forces developed in the columns of plane frames. Unbraced frames are likely to sway producing lateral displacements. Axial forces acting through these displacements produce secondary moments and these should be included in the structural analysis. AISC ASD recommends a magnification factor to amplify the moments obtained from a linear analysis. On the other hand, LRFD recommends two procedures to account for these additional moments: the first procedure uses an amplification factor as described in Chapter 3 under requirements for compression members; the other, used in this study, is based on a non- linear analysis of the structure. Because a non-linear analysis is expensive, an approximate method was used instead.

The paper by Wood, Beaulieu, and Adams [26] presents an approximate technique by modifying the first-order results. The initial step in the modification procedure is to compute the sway forces. The factored loads are applied to the structure, and the lateral displacements, denoted by $\Delta_i$ are computed by first-order theory. The displacements, $\Delta_i$, are computed by averaging the lateral displacements of the joints at each floor level, $i$. The additional story shears due to vertical loads are computed:

$$V_i' = \frac{\Sigma P_i}{h_i}(\Delta_{i+1} - \Delta_i) \qquad [5.1]$$

in which $V_i'$ = additional shear in story i due to sway forces; $\Sigma P_i$ = sum of the column axial loads in story i; $h_i$ = height of story i; and $\Delta_{i+1}$, $\Delta_i$ = displacements of level $i + 1$ *and* $i$, respectively.

The sway forces due to vertical loads, $H_i'$, are then computed as the difference between the additional story shears at each level,

$$H_i' = V_{i-1}' - V_i' \qquad [5.2]$$

The sway forces, $H_i'$, are added to the applied lateral loads, and the structure reanalysed. When the $\Delta_i$s at the end of a cycle are equal to the ones in the previous cycle, the method has converged, and the resulting forces and moments include the P-$\Delta$ effects. This method is applicable to both combined loads (wind, dead and live) and vertical loads. In the latter case, since a linear analysis does not reflect any lateral displacements, it is necessary to assume an initial imperfection in each story. Initial lateral forces are computed on the basis of a story sway, $\Delta/L = 0.002$, which is a common erection tolerance. Initial sway forces are computed based on this initial imperfection and are applied to the structure as horizontal joint forces. Then, the procedure for including P-$\Delta$ effects is repeated.

## 5.2 Comparison of Results

The results obtained from this study are listed in Tables 3, 4, 5 and 6. Each table contains three columns corresponding to the member number, the ASD design solution, and the LRFD design solution for that member. The total steel weight required by each criteria is listed at the bottom of each table, and the percentage shown corresponds to the percentage difference of steel weight between the two design solutions.

Table 3 shows the design solutions for the two-story building. Both LRFD and ASD resulted in the same member sizes for the columns. For the beams, however, an 11% saving in steel weight was obtained using the LRFD criteria. The total steel weight of the two-story frame designed by LRFD was 7.3% less than the ASD design solution.

Table 4 and Table 5 show design solutions for the five-story building. The only difference between the tables is the depth of the section considered in the design of the beams. Table 4 shows designs with only W18 sections for beams, while Table 5 shows designs with W21 sections for beams. These tables show the effect of increasing beam depth on the overall design solutions. While an increase in depth did not change the size of the columns significantly, it did increase the weight of the beams. Therefore, W18 sections were considered to be a better choice for the design of beams.

From Table 4, it can be seen that for the five-story frame there is a saving of steel weight of 11% for beams designed by the LRFD criteria. Note that this is similar to the savings in beam weight obtained for the two-story frame. Using LRFD also resulted in savings of 4.3% in the total steel weight of columns with the overall saving in total structural steel at 8.6%.

The results of the design of the ten-story frame are shown in Table 6. In this case, the design solution by the LRFD criteria resulted in a frame that was 7.8% lighter in steel weight than that designed by the ASD method. The total weight of the columns, however, was 6% lighter in the LRFD design solution, which is a little higher than for the five-story frame.

In general, both methods resulted in similar design solutions with LRFD tending to give lighter members than ASD. In cases where LRFD resulted in a smaller member than ASD, it was only one or two sizes down in the table of W-sections as listed in the AISC Manual [2]. In this manual, the nominal weight per linear foot does not vary proprotionally; therefore, there is a difference in saving percentage. Consequently, the total saving percentage of the structures are somewhat different.

The increase in savings in total steel weight for columns (0%, 4.3% and 6%) for the two, five and ten-story buildings, respectively, may be due partly to the procedure used in the computation of P-$\Delta$ effects as specified in LRFD. The moments obtained from this analysis are not multiplied by a magnification factor as is done in the ASD method. ASD Specifications require that all moments obtained from a linear analysis and induced in compression members, be multiplied by a factor $0.85/(1 - f_a/F_e)$ to account for sidesway effects. This often unnecessarily increases gravity moments that are not associated with sidesway. Since moments due to P-$\Delta$ effects were computed, it was not necessary to multiply these by an amplification factor. The fact that the savings in steel increased with the height of the buildings is the result of an increase of P-Δ effect. For example, the P-Δ effects in the ten-story building are greater than that for the two-story building.

The economies found in both beams and columns arose as a result of using load factors. The ratio of live to dead load used in the three designs was about 0.6. Therefore, the load combination that governed the design in most cases was $1.2W_D + 1.6W_L$. It was found that wind loads were relatively small, so that the load effects produced by the load combinations that contained wind loads did not exceeded the live plus dead load combination. While Allowable Stress Design does not distinguish between dead and live loads, LRFD criteria recognizes the differences in variability of these loads. Therefore, the design solutions using ASD method tended to give an overall heavier structure.

It was found that once the factored loads are determined for one member, the two design methods are of about equal complexity. For large structures, computer programs are usually used to compute load effects in individual members. These can be executed using the factored loads instead of the nominal loads, so that the load effects obtained are already factored. One drawback of the LRFD procedure is that it requires a non-linear analysis for the computation of moments applied on compression members, which include P-Δ. A non-linear analysis might become very costly and time-consuming, and it is unlikely that medium-sized companies will have access to computer programs with this capability. For tall buildings, a limit of drift for serviceability consideration is usually required. Because an approximate method of including sidesway effects allows the user to have more control of lateral deflections, LRFD recommends it over an exact solution [18]. An advantage of an approximate method is that a linear analysis can be used for the computation of load effects, and its technique is easy to understand and perform.

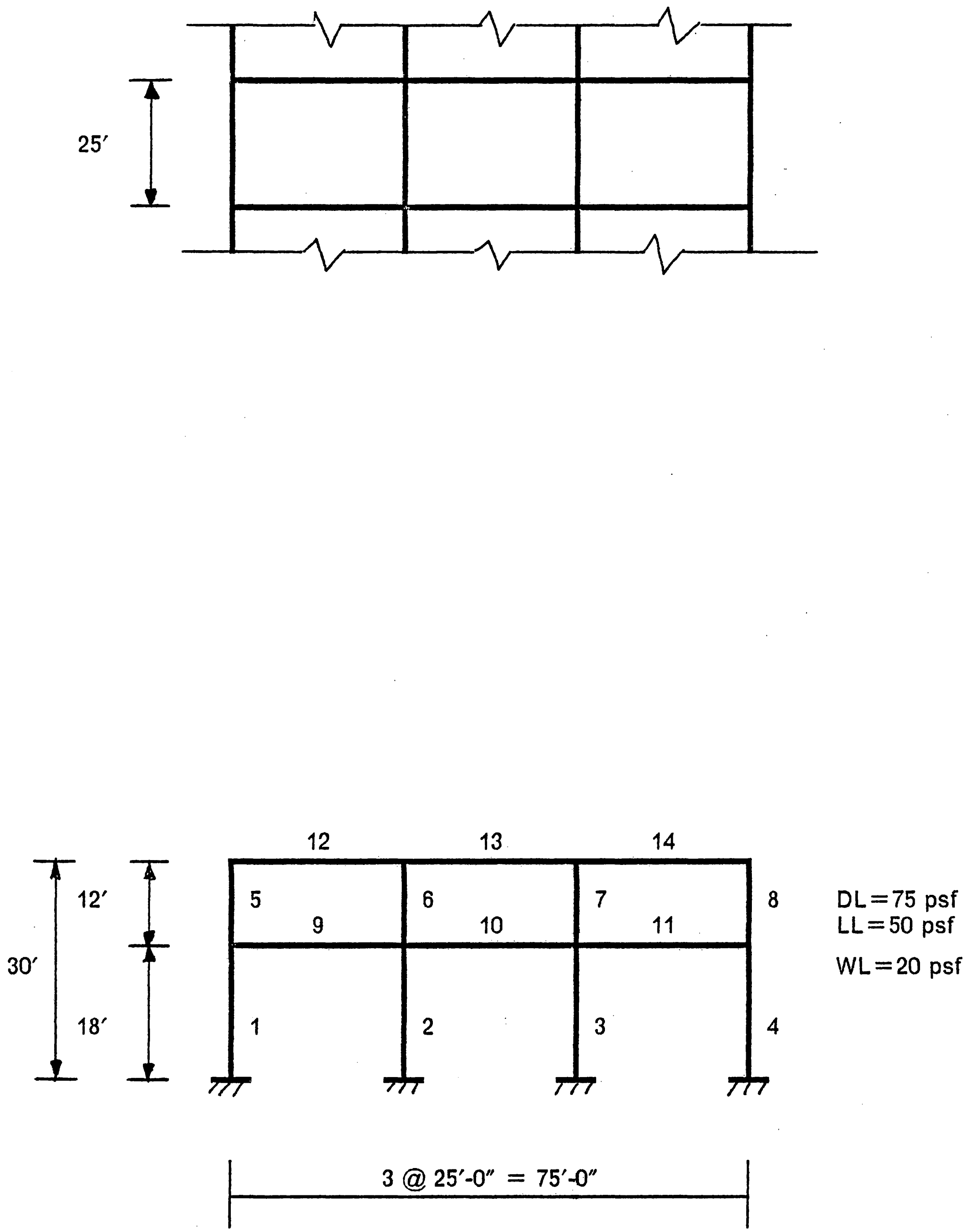



**Figure 8. Plan and elevation of two story building**

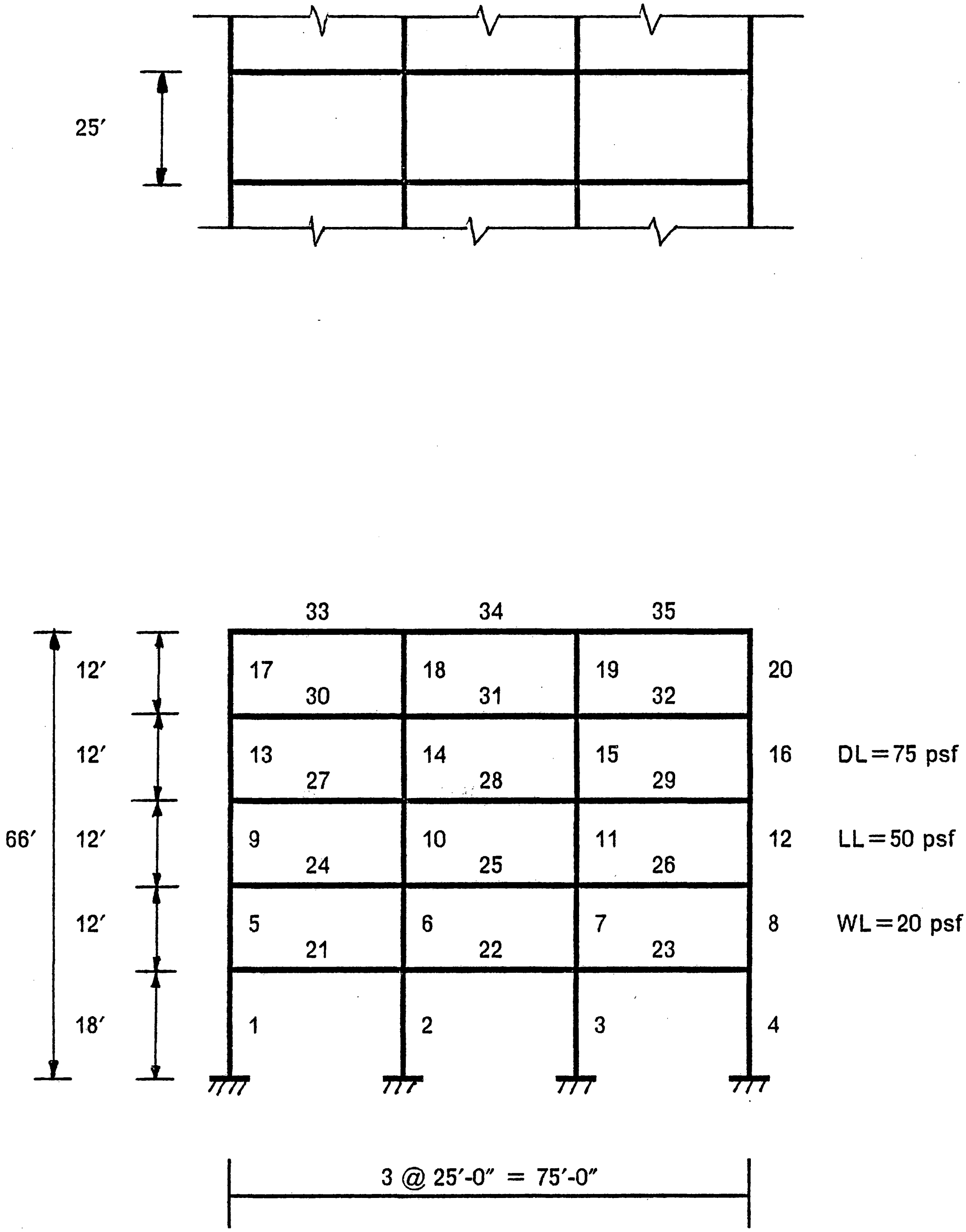



**Figure 9. Plan and elevation of five-story building**

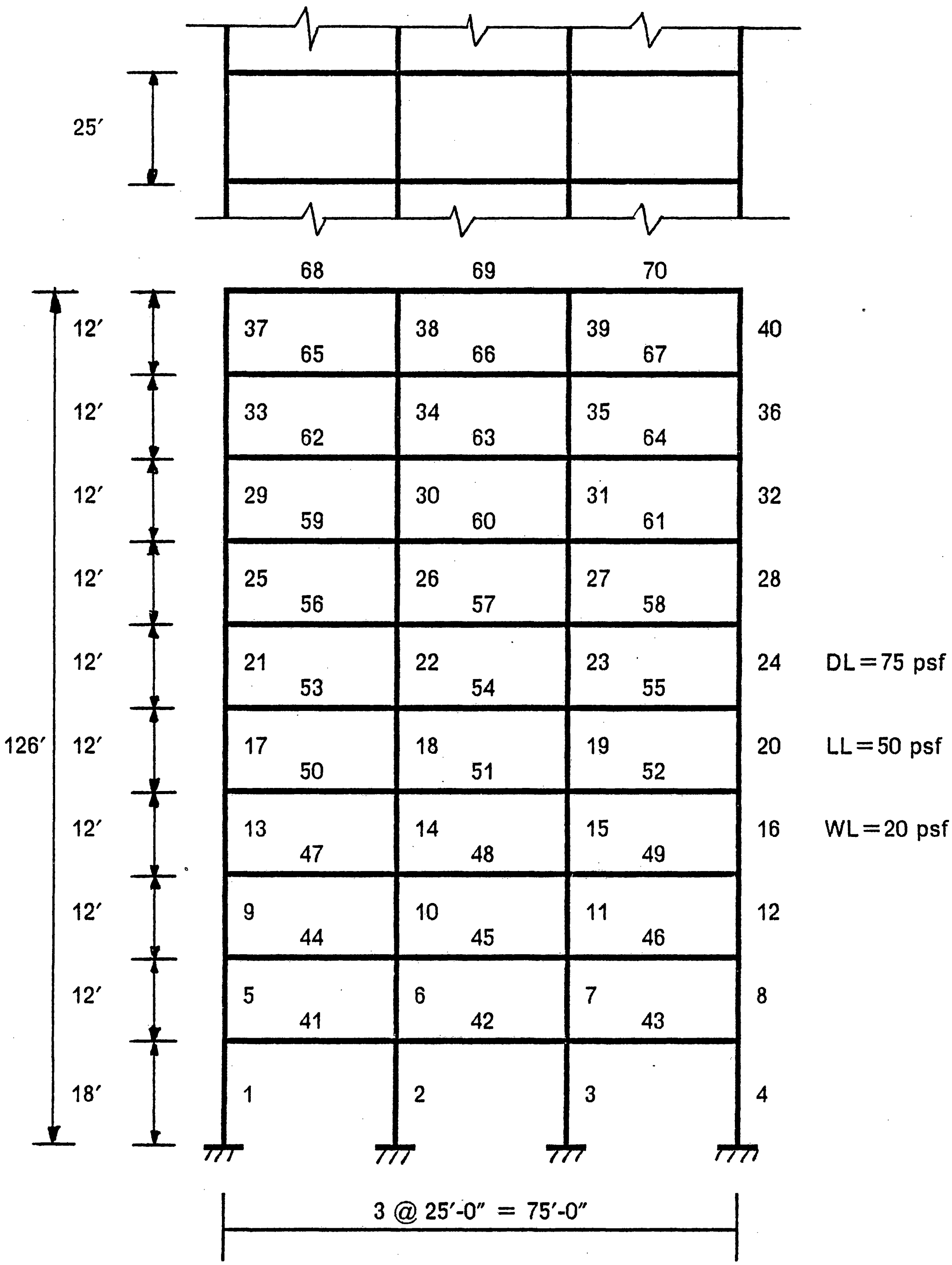



**Figure 10. Plan and elevation of ten-story building**

Table 3. **Design solutions for two-story building using two methods**

| MEMBER # | ASD | LRFD |
|---|---|---|
| 1 | W14X43 | W14X43 |
| 2 | W14X53 | W14X53 |
| 3 | W14X53 | W14X53 |
| 4 | W14X43 | W14X43 |
| 5 | W14X34 | W14X34 |
| 6 | W14X30 | W14X30 |
| 7 | W14X30 | W14X30 |
| 8 | W14X34 | W14X34 |
| 9 | W18X76 | W18X65 |
| 10 | W18X76 | W18X71 |
| 11 | W18X76 | W18X65 |
| 12 | W18X71 | W18X65 |
| 13 | W18X76 | W18X71 |
| 14 | W18X71 | W18X65 |
| | COLUMNS = 4992 LB | COLUMNS = 4992 LB (0%) |
| | BEAMS = 11150 LB | BEAMS = 10050 LB (11%) |
| | TOTAL = 16142 LB | TOTAL = 15042 LB (7.3%) |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

**Table 4. Design solutions for five-story building using two methods**

| MEMBER # | ASD | LRFD |
|---|---|---|
| 1 | W14X61 | W14X61 |
| 2 | W14X90 | W14X90 |
| 3 | W14X90 | W14X90 |
| 4 | W14X61 | W14X61 |
| 5 | W14X61 | W14X53 |
| 6 | W14X68 | W14X61 |
| 7 | W14X68 | W14X61 |
| 8 | W14X61 | W14X53 |
| 9 | W14X48 | W14X43 |
| 10 | W14X53 | W14X53 |
| 11 | W14X53 | W14X53 |
| 12 | W14X48 | W14X43 |
| 13 | W14X38 | W14X38 |
| 14 | W14X43 | W14X43 |
| 15 | W14X43 | W14X43 |
| 16 | W14X38 | W14X38 |
| 17 | W14X43 | W14X38 |
| 18 | W14X30 | W14X30 |
| 19 | W14X30 | W14X30 |
| 20 | W14X43 | W14X38 |

ASD = Allowble Stress Design

LRFD = Load Resistance Factor Design

Table 4. Design solutions for five-story building using two methods (continued)

| MEMBER # | ASD | LRFD |
|---|---|---|
| 21 | W18X76 | W18X65 |
| 22 | W18X76 | W18X71 |
| 23 | W18X76 | W18X65 |
| 24 | W18X76 | W18X71 |
| 25 | W18X76 | W18X71 |
| 26 | W18X76 | W18X71 |
| 27 | W18X76 | W18X65 |
| 28 | W18X76 | W18X71 |
| 29 | W18X76 | W18X65 |
| 30 | W18X76 | W18X65 |
| 31 | W18X76 | W18X71 |
| 32 | W18X76 | W18X65 |
| 33 | W18X71 | W18X65 |
| 34 | W18X76 | W18X71 |
| 35 | W18X71 | W18X65 |
| | COLUMNS = 14652 LB | COLUMNS = 14052 LB (4.3%) |
| | BEAMS = 28250 LB | BEAMS = 25425 LB (11%) |
| | TOTAL = 42902 LB | TOTAL = 25425 LB (8.6%) |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

**Table 5. Design solutions for five-story building, using two methods and W21 sections for the beams**

| MEMBER # | ASD | LRFD |
|---|---|---|
| 1 | W14X61 | W14X61 |
| 2 | W14X90 | W14X90 |
| 3 | W14X90 | W14X90 |
| 4 | W14X61 | W14X61 |
| 5 | W14X61 | W14X53 |
| 6 | W14X68 | W14X61 |
| 7 | W14X68 | W14X61 |
| 8 | W14X61 | W14X53 |
| 9 | W14X48 | W14X43 |
| 10 | W14X53 | W14X53 |
| 11 | W14X53 | W14X53 |
| 12 | W14X48 | W14X43 |
| 13 | W14X38 | W14X38 |
| 14 | W14X43 | W14X43 |
| 15 | W14X43 | W14X43 |
| 16 | W14X38 | W14X38 |
| 17 | W14X43 | W14X38 |
| 18 | W14X30 | W14X30 |
| 19 | W14X30 | W14X30 |
| 20 | W14X43 | W14X38 |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

Table 5. Design solutions for five-story building, using two methods and W21 sections for the beams (continued)

| MEMBER # | ASD | LRFD |
|---|---|---|
| 21 | W21X83 | W21X68 |
| 22 | W21X83 | W21X68 |
| 23 | W21X83 | W21X68 |
| 24 | W21X83 | W21X68 |
| 25 | W21X83 | W21X68 |
| 26 | W21X83 | W21X68 |
| 27 | W21X73 | W21X68 |
| 28 | W21X83 | W21X68 |
| 29 | W21X73 | W21X68 |
| 30 | W21X73 | W21X68 |
| 31 | W21X83 | W21X68 |
| 32 | W21X73 | W21X68 |
| 33 | W21X73 | W21X62 |
| 34 | W21X83 | W21X68 |
| 35 | W21X73 | W21X62 |
| | COLUMNS = 14652 lb | COLUMNS = 14052 lb (4.2%) |
| | BEAMS = 29625 lb | BEAMS = 25200 lb (17%) |
| | TOTAL = 44277 lb | TOTAL = 39252 lb (12.8%) |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

**Table 6. Design solutions for ten-story building using two methods**

| MEMBER # | ASD | LRFD |
|---|---|---|
| 1 | W14X90 | W14X90 |
| 2 | W14X159 | W14X145 |
| 3 | W14X159 | W14X145 |
| 4 | W14X90 | W14X90 |
| 5 | W14X90 | W14X90 |
| 6 | W14X132 | W14X120 |
| 7 | W14X132 | W14X120 |
| 8 | W14X90 | W14X90 |
| 9 | W14X90 | W14X82 |
| 10 | W14X120 | W14X109 |
| 11 | W14X120 | W14X109 |
| 12 | W14X90 | W14X82 |
| 13 | W14X82 | W14X74 |
| 14 | W14X99 | W14X99 |
| 15 | W14X99 | W14X99 |
| 16 | W14X82 | W14X74 |
| 17 | W14X74 | W14X68 |
| 18 | W14X90 | W14X90 |
| 19 | W14X90 | W14X90 |
| 20 | W14X74 | W14X68 |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

Table 6. Design solutions for ten-story building using two methods (continued)

| MEMBER # | ASD | LRFD |
|---|---|---|
| 21 | W14X68 | W14X61 |
| 22 | W14X82 | W14X74 |
| 23 | W14X82 | W14X74 |
| 24 | W14X68 | W14X61 |
| 25 | W14X61 | W14X61 |
| 26 | W14X68 | W14X61 |
| 27 | W14X68 | W14X61 |
| 28 | W14X61 | W14X61 |
| 29 | W14X53 | W14X48 |
| 30 | W14X53 | W14X53 |
| 31 | W14X53 | W14X53 |
| 32 | W14X53 | W14X48 |
| 33 | W14X43 | W14X38 |
| 34 | W14X43 | W14X43 |
| 35 | W14X43 | W14X43 |
| 36 | W14X43 | W14X38 |
| 37 | W14X43 | W14X43 |
| 38 | W14X30 | W14X30 |
| 39 | W14X30 | W14X30 |
| 40 | W14X43 | W14X43 |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

Table 6. Design solutions for ten-story building using two methods (continued)

| MEMBER # | ASD | LRFD |
|---|---|---|
| 41 | W18X76 | W18X71 |
| 42 | W18X76 | W18X71 |
| 43 | W18X76 | W18X71 |
| 44 | W18X76 | W18X71 |
| 45 | W18X76 | W18X71 |
| 46 | W18X76 | W18X71 |
| 47 | W18X76 | W18X71 |
| 48 | W18X76 | W18X71 |
| 49 | W18X76 | W18X71 |
| 50 | W18X76 | W18X71 |
| 51 | W18X76 | W18X71 |
| 52 | W18X76 | W18X71 |
| 53 | W18X76 | W18X71 |
| 54 | W18X76 | W18X71 |
| 55 | W18X76 | W18X71 |
| 56 | W18X76 | W18X71 |
| 57 | W18X76 | W18X71 |
| 58 | W18X76 | W18X71 |
| 59 | W18X76 | W18X65 |
| 60 | W18X76 | W18X71 |

ASD = Allowable Stress Design

LRFD = Load Resistance Factor Design

Table 6. Design solutions for ten-story building using two methods (continued)

| MEMBER # | ASD | LRFD |
|---|---|---|
| 61 | W18X76 | W18X65 |
| 62 | W18X76 | W18X65 |
| 63 | W18X76 | W18X71 |
| 64 | W18X76 | W18X65 |
| 65 | W18X76 | W18X65 |
| 66 | W18X76 | W18X71 |
| 67 | W18X76 | W18X65 |
| 68 | W18X71 | W18X65 |
| 69 | W18X76 | W18X71 |
| 70 | W18X71 | W18X65 |
| | COLUMNS=40668 lb | COLUMNS=38316 lb (6%) |
| | BEAMS=56750 lb | BEAMS=52050 lb (9%) |
| | TOTAL=97418 lb | TOTAL=90366 lb (7.8%) |

ASD=Allowable Stress Design

LRFD=Load Resistance Factor Design

# Chapter VI

# Summary and Conclusions

After many years of research, the American Institute of Steel Construction has published the Load and Resistance Factor Design Specifications. It is based on the concept of limit states and uses load and resistance factors to account for the uncertainties in loads and material properties. These specifications were developed from a reliability study.

The design requirements for LRFD for steel frames are presented. Three office buildings of two, five and ten stories are designed using LRFD criteria for beams, columns and beam-columns. To accomplish this, a computer program was developed. The program consists of three modules which perform the structural analysis, the design of beams, and the design of columns and beam-columns. These modules are linked together so that the analysis and design processes is completely automated. Finally, the design solutions obtained from the program's execution are compared to solutions using the current Allowable Stress Design criteria.

The following conclusions were drawn from this study:

1. The proposed Load and Resistance Factor Design method yields a structure comparable to that designed by the Allowable Stress Design method

with a moderate saving in steel weight of 8% for the design examples and ratios of live to dead load used in this study. These economies are due to the use of different load factors which reflects the effects of different type of loads in a more realistic manner. Note that for other ratios of live to dead load the results may be different.

2. After factored loads are computed, LRFD proved to be of equal complexity as ASD. The procedures were similar in terms of effort and number of calculations.
3. The use of load and resistance factors provide the designer with a better understanding of the behavior of the structure under the influence of different load conditions.

The following topics required further investigation. LRFD recommends two procedures to find the required flexural strength in beam-columns. Since only one of them was used in this study, the significance of using the other method can be evaluated. A comparison of the same kind presented here can be made for other structural components, such as fixed and pin connections, plate girders and composite structures. Additional computer programs can be develop for the design of these structural components according to LRFD Specifications.

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# Appendix A

# Definition of Program Variables

**A:** clear distance between transverse stiffeners (in).

**AG:** cross-sectional area of member ($in^2$).

**AWEB:** area of the web = D*TW ($in^2$).

**BF:** flange width (in).

**CB:** bending coefficient dependent upon moment gradient.

**CB1:** possible value for coefficient CB.

**CB2:** possible value for coefficient CB.

**CW:** warping torsional constant ($in^6$).

**D:** depth of member (in).

**DAF:** depth of member / area of flange (1/in).

**DMAX:** maximum beam depth (in).

**DMIN:** minimum beam depth (in).

**E:** modulus of elasticity of steel (ksi).

**FRC:** critical stress (ksi).

**FY:** yield stress of steel (ksi).

**G:** shear modulus of elasticity of steel (ksi).

**H:** twice the distance between the neutral axis and the center of gravity of the compression flange (in).

**HC:** twice the distance between the neutral axis and the inside face of the compression flange less fillet or radius (in).

**INTERACT:** interaction equation for axial compression and flexure.

**INTERMIN:** value of the interaction equation for the lightest satisfactory member.

**IX:** moment of inertia about the major axis ($in^4$).

**IY:** moment of inertia about the minor axis ($in^4$).

**JT:** torsional constant ($in^4$)

**K:** effective length factor.

**KF:** distance between the outside of compression flange and the fillet or radius (in).

**KV:** web plate buckling coefficient.

**L:** member length (ft).

**LAMBDAC:** column slenderness parameter.

**LB:** laterally unbraced length (ft).

**LFLB:** BF/2TW; flange width-thickness ratio for flange local buckling.

**LFLBC:** BF/2TW; flange width-thickness ratio for compression.

**LLTB:** LB/RY; minor axis slenderness ratio for lateral torsional buckling.

**LPFLB:** largest value of LFLB for which MN=MP.

**LPLTB:** largest value of LLTB for which MN=MP.

**LPWLB:** largest value of LWLB for which MN=MP.

**LRFLB:** largest value of LFLB for which buckling is inelastic.

**LRFLBC:** limiting flange width-thickness ratio for compression.

**LRLTB:** largest value of LLTB for which buckling is inelastic.

**LRWLB:** largest value of LWLB for which buckling is inelastic.

**LRWLBC:** limiting web width-thickness ratio for compression.

**LWLB:** HC/TW; web width-thickness ratio for web local buckling.

**LWLBC:** HC/TW; web width-thickness ratio for compression.

**M1:** MA or MB from moment diagram (k-ft).

**M2:** MA of MB from moment diagram (k-ft).

**M11:** MA or MC from moment diagram (k-ft).

**M12:** MA or MC from moment diagram (k-ft).

**M21:** MB or MC from moment diagram (k-ft).

**M22:** MB or MC from moment diagram (k-ft).

**MA:** left-end moment due to critical load combination (k-ft).

**MB:** right-end moment due to critical load combination (k-ft).

**MC:** moment at bracing due to critical load combination.

**MN:** nominal flexural moment strength of member (k-ft).

**MNFLB:** nominal moment strength for the limit state of flange local buckling (k-ft).

**MNLTB:** nominal moment strength for the limit state of lateral torsional buckling (k-ft).

**MNWLB:** nominal moment strength for the limit state of web local buckling (k-ft).

**MNX:** nominal flexural strength about major axis (k-ft).

**MNY:** nominal flexural strength about minor axis (k-ft).

**MP:** fully plastic moment of member (k-ft).

**MPX:** plastic moment about major axis (k-ft).

**MPY:** plastic moment about minor axis (k-ft).

**MRFLB:** limiting flange local buckling moment (k-ft).

**MRLTB:** limiting lateral torsional buckling moment (k-ft).

**MRWLB:** limiting web local buckling moment (k-ft).

**MUX:** required flexural strength about major axis (k-ft).

**MUY:** required flexural strength about minor axis (k-ft).

**MUMAX:** maximum factored moment due to critical load combination.

**N:** number of segments.

**ND:** nominal depth of member (in).

**NDMIN:** nominal depth of lightest satisfactory member (in).

**PHIB:** resistance factor for flexure = 0.9

**PHIC:** resistance factor for compression = 0.85

**PHIMN:** moment resistance of member (k-ft).

**PHIMNX:** moment resistance of member about major axis (k-ft).

**PHIMNY:** moment resistance of member about minor axis (k-ft).

**PHIPN:** compressive resistance of member (kips).

**PHIV:** resistance factor for shear = 0.9

**PHIVN:** shear resistance of member (kips)

**PI:** circumference of circle divided by its diameter.

**RT:** radius of gyration of a section, comprising the compression flange, 1/3 of the compression web area, taken about an axis in the plane of the web (in).

**RX:** radius of gyration with respect to the major axis (in).

**RY:** radius of gyration with respect to the minor axis (in).

**SX:** elastic section modulus about the major axis ($in^3$).

**SY:** elastic section modulus about the minor axis ($in^3$).

**TF:** flange thickness (in).

**TW:** web thickness (in).

**VN:** monimal shear strength of the member (kips)

**VUMAX:** maximum factored shear due to critical loading combination (kips).

**WT:** nominal weight of member (lb/ft).

**WTMIN:** weight of lightest satisfactory member (lb/ft).

**ZX:** plastic section modulus about major axis ($in^3$).

**ZY:** plastic section modulus about minor axis ($in^3$).

## APPENDIX B. LISTING OF COMPUTER PROGRAM

```
*************************************************************************
*                                                                       *
*                          Module DESBEAM                               *
*                                                                       *
*************************************************************************

50065 REM-------------------------------------------------------------
50070 REM                          READ BEAMS
50075 REM-------------------------------------------------------------
50080 REM READ INFORMATION CONTAINED IN DATA FILE "BEAMF.DAT", WHICH WILL
50085 REM BE USED IN THE DESIGN OF BEAMS. READ FOR EACH MEMBER UNIFORM
50090 REM AND CONCENTRATED LOADS (MN,MAT,ACT,DIST), LENGTH AND END FORCES
50095 REM FOR EACH LOAD CONDITION.
50100 REM
50105 REM
50110 REM
50115 COLOR 1,5,3
50120 OPTION BASE 1
50125 CLS:KEY OFF
50130 OPEN "BEAMF.DAT" FOR INPUT AS #2
50135 INPUT #2,NE
50140 N=20
50145 DIM ELENG(NE),R11(NE),R12(NE),R21(NE),R22(NE),R31(NE),R32(NE),R41(NE)
50150 DIM R42(NE),R51(NE),R52(NE),M11(NE),M12(NE),M21(NE),M22(NE),M31(NE),
M32(NE)
50155 DIM M41(NE),M42(NE),M51(NE),M52(NE)
50160 DIM R1(NE),R2(NE),R3(NE),R4(NE),R5(NE)
50165 DIM M1(NE),M2(NE),M3(NE),M4(NE),M5(NE)
50170 DIM VMAX(NE),MMAX(NE),LB(NE),MA(NE),MB(NE),NDMIN(NE),WTMIN(NE),
IXMIN(NE)
50175 DIM IYMIN(NE),CODE(NE),LCOM(NE)
50180 DIM W1(NE),W2(NE),W3(NE),W4(NE),W5(NE),A1(NE),A2(NE),A3(NE),
A4(NE),A5(NE),B1(NE),B2(NE),B3(NE),B4(NE),B5(NE),P1(NE,20),P2(NE,20),
P3(NE,20),P4(NE,20),P5(NE,20)
50185 DIM C1(NE,20),C2(NE,20),C3(NE,20),C4(NE,20),C5(NE,20),MPREV(NE),
P(30),C(30),MC(NE),X(N+1),M(N+1),V(N+1),N21(NE),N22(NE),N23(NE),N24(NE),
N25(NE)
50190 INPUT #2,I,ELENG,LB
50195 WHILE I <> 0
50200       CODE(I)=1
50205       ELENG(I)=ELENG : LB(I)=LB
50210       INPUT #2,I,ELENG,LB
50215 WEND
50220 IF EOF(2) THEN 50295
50225 INPUT #2,LC
50230 INPUT #2,MN,MAT,ACT,DIST
```

```
50235 WHILE MN <> 0
50240       I=MN
50245       ON LC GOSUB 50300,50340,50380,50420,50460
50250       INPUT #2,MN,MAT,ACT,DIST
50255 WEND
50260 INPUT #2,BEAM,F1,F2,F3,F4,F5,F6
50265 WHILE BEAM <> 0
50270       I=BEAM
50275       ON LC GOSUB 50505,50535,50565,50590,50620
50280       INPUT #2,BEAM,F1,F2,F3,F4,F5,F6
50285 WEND
50290 GOTO 50220
50295 RETURN
50300 '
50305 'SUBROUTINE FOR LC=1
50310 IF MAT=1 THEN 50315 ELSE 50325
50315 N21(I)=N21(I) + 1 : N21=N21(I)
50320 P1(I,N21)=-ACT : C1(I,N21)=DIST
50325 IF MAT=2 THEN 50330 ELSE 50335
50330 W1(I)=-ACT : A1(I)=DIST : B1(I)=ELENG(I)
50335 RETURN
50340 '
50345 'SUBROUTINE FOR LC=2
50350 IF MAT=1 THEN 50355 ELSE 50365
50355 N22(I)=N22(I) + 1 : N22=N22(I)
50360 P2(I,N22)=-ACT : C2(I,N22)=DIST
50365 IF MAT=2 THEN 50370 ELSE 50375
50370 W2(I)=-ACT : A2(I)=DIST : B2(I)=ELENG(I)
50375 RETURN
50380 '
50385 'SUBROUTINE FOR LC=3
50390 IF MAT=1 THEN 50395 ELSE 50405
50395 N23(I)=N23(I) + 1 : N23=N23(I)
50400 P3(I,N23)=-ACT : C3(I,N23)=DIST
50405 IF MAT=2 THEN 50410 ELSE 50415
50410 W3(I)=-ACT : A3(I)=DIST : B3(I)=ELENG(I)
50415 RETURN
50420 '
50425 'SUBROUTINE FOR LC=4
50430 IF MAT=1 THEN 50435 ELSE 50445
50435 N24(I)=N24(I) + 1 : N24=N24(I)
50440 P4(I,N24)=-ACT : C4(I,N24)=DIST
50445 IF MAT=2 THEN 50450 ELSE 50455
50450 W4(I)=-ACT : A4(I)=DIST : B4(I)=ELENG(I)
50455 RETURN
50460 '
50465 'SUBROUTINE FOR LC=5
50470 IF MAT=1 THEN 50475 ELSE 50485
50475 N25(I)=N25(I) + 1 : N25=N25(I)
50480 P5(I,N25)=-ACT : C5(I,N25)=DIST
50485 IF MAT=2 THEN 50490 ELSE 50495
```

```
50490 W5(I)=-ACT : A5(I)=DIST : B5(I)=ELENG(I)
50495 RETURN
50500 REM
50505 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
50510 REM SUBROUTINE FOR LC=1 : END FORCES DUE TO LOAD COMBINATION 1
50515 REM
50520 R11(I)=F2 : R12(I)=F5 : M11(I)=-F3 : M12(I)=F6
50525 RETURN
50530 REM
50535 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
50540 REM SUBROUTINE FOR LC=2 : END FORCES DUE TO LOAD COMBINATION 2
50545 REM
50550 R21(I)=F2 : R22(I)=F5 : M21(I)=-F3 : M22(I)=F6
50555 RETURN
50560 REM
50565 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
50570 REM SUBROUTINE FOR LC=3 : END FORCES DUE TO LOAD COMBINATION 3
50575 REM
50580 R31(I)=F2 : R32(I)=F5 : M31(I)=-F3 : M32(I)=F6
50585 RETURN
50590 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
50595 REM SUBROUTINE FOR LC=4 : END FORCES DUE TO LOAD COMBINATION 4
50600 REM
50605 R41(I)=F2 : R42(I)=F5 : M41(I)=-F3 : M42(I)=F6
50610 RETURN
50615 REM
50620 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
50625 REM SUBROUTINE FOR LC=5 : END FORCES DUE TO LOAD COMBINATION 5
50630 REM
50635 R51(I)=F2 : R52(I)=F5 : M51(I)=-F3 :M52(I)=F6
50640 RETURN
50645 REM
50650 REM
50655 REM--------------------------------------------------------------
50660 REM                              ANALYSIS
50665 REM--------------------------------------------------------------
50670 REM ANALYSIS SUBROUTINE TO CALCULATE MAXIMUM FACTORED MOMENTS AND
SHEARS
50675 REM FOR EACH BEAM.
50680 REM
50685 FOR I=1 TO NE
50690     IF CODE(I) <> 1 THEN 51160
50695     CLS: PRINT TAB(15) "Program is calculating maximum factored"
50700     PRINT TAB(15) "shear and moment for member ";I
50705     PRINT TAB(20) "one moment please..."
50710 '
50715 REM------------ LOAD COMBINATION #1 ---------------------------
50720 V1=R11(I)
50725 M1=M11(I)
50730 L=ELENG(I)
50735 J%=(LB(I)*N)/L + 1
```

```
50740 N1=1 : W(1)=W1(I) : A(1)=A1(I) : B(1)=B1(I)
50745 N2=N21(I)
50750 FOR K=1 TO N2
50755     P(K)=P1(I,K)
50760     C(K)=C1(I,K)
50765 NEXT K
50770 MMAX(I)=ABS(M1) : VMAX(I)=ABS(V1)
50775 GOSUB 51835 : 'SUBROUTINE VMMAX
50780 V2=R12(I)
50785 M2=M12(I)
50790 MA(I)=M1 : MB(I)=M2 : MC(I)=MLB
50795 LCOM(I)=1
50800 MPREV(I)=MMAX(I)
50805 REM ------------------ LOAD COMBINATION #2 -----------------------
50810 V1=R21(I)
50815 M1=M21(I)
50820 N1=1 : W(1)=W2(I) : A(1)=A2(I) : B(1)=B2(I)
50825 N2=N22(I)
50830 FOR K=1 TO N2
50835     P(K)=P2(I,K)
50840     C(K)=C2(I,K)
50845 NEXT K
50850 GOSUB 51835 : 'SUBROUTINE VMMAX
50855 V2=R22(I)
50860 M2=M22(I)
50865 IF MMAX(I) > MPREV(I) THEN 50870 ELSE 50880
50870    LCOM(I)=2 :MPREV(I)=MMAX(I)
50875    MA(I)=M1 : MB(I)=M2 : MC(I)=MLB
50880 REM ------------------- LOAD COMBINATION #3 ----------------------
50885 V1=R31(I)
50890 M1=M31(I)
50895 N1=1 : W(1)=W3(I) : A(1)=A3(I) : B(1)=B3(I)
50900 N2=N23(I)
50905 FOR K=1 TO N2
50910     P(K)=P3(I,K)
50915     C(K)=C3(I,K)
50920 NEXT K
50925 GOSUB 51835 : 'SUBROUTINE VMMAX
50930 V2=R32(I)
50935 M2=M32(I)
50940 IF MMAX(I) > MPREV(I) THEN 50945 ELSE 50955
50945    LCOM(I)=3 : MPREV(I)=MMAX(I)
50950    MA(I)=M1 : MB(I)=M2 : MC(I)=MLB
50955 REM --------------------- LOAD COMBINATION #4 -------------------
50960 V1=R41(I)
50965 M1=M41(I)
50970 N1=1 : W(1)=W4(I) : A(1)=A4(I) : B(1)=B4(I)
50975 N2=N24(I)
50980 FOR K=1 TO N2
50985     P(K)=P4(I,K)
50990     C(K)=C4(I,K)
```

```
50995 NEXT K
51000 GOSUB 51835: 'SUBROUTINE VMMAX
51005 V2=R42(I)
51010 M2=M42(I)
51015 IF MMAX(I) > MPREV(I) THEN 51020 ELSE 51030
51020    LCOM(I)=4 : MPREV(I)=MMAX(I)
51025    MA(I)=M1 : MB(I)=M2 : MC(I)=MLB
51030 REM ------------------- LOAD COMBINATION #5 --------------------
51035 V1=R51(I)
51040 M1=M51(I)
51045 N1=1 : W(1)=W5(I) : A(1)=A5(I) : B(1)=B5(I)
51050 N2=N25(I)
51055 FOR K=1 TO N2
51060     P(K)=P5(I,K)
51065     C(K)=C5(I,K)
51070 NEXT K
51075 GOSUB 51835 : 'SUBROUTINE VMMAX
51080 V2=R52(I)
51085 M2=M52(I)
51090 IF MMAX(I) > MPREV(I) THEN 51095 ELSE 51105
51095    LCOM(I)=5 : MPREV(I)=MMAX(I)
51100    MA(I)=M1 : MB(I)=M2 : MC(I)=MLB
51105 '
51110 '
51115 CLS : LOCATE 13,8
51120 PRINT "MAXIMUM FACTORED FORCES FOR MEMBER ";I
51125 PRINT TAB(8) "FOR LOAD COMBINATION #";LCOM(I)
51130 PRINT TAB(8) "MAXIMUM FACTORED SHEAR = ";VMAX(I)
51135 PRINT TAB(8) "MAXIMUM FACTORED MOMENT = ";MMAX(I)
51140 LOCATE 25
51145 PRINT TAB(25) "PRESS <ENTER> TO CONTINUE"
51150 H$=INKEY$ : IF H$="" THEN 51150
51155 IF ASC(H$) <> 13 THEN 51150
51160 NEXT I
51165 RETURN
51170 REM-------------------------------------------------------------
51175 REM                         DESIGN
51180 REM-------------------------------------------------------------
51185 REM DESIGN SUBROUTINE TO PICK BEST W-SECTION THAT MEETS ALL
REQUIREMENTS
51190 REM FOR EACH BEAM.
51195 REM
51200 REM
51205 PI=3.141592654#
51210 E=29000
51215 G=11200
51220 PHIV=.9
51225 PHIB=.9
51230 DMAX=36
51235 DMIN=4
51240 FY=36
```

```
51245 FOR I=1 TO NE
51250     IF CODE(I) <> 1  THEN 51315
51255     CLS: PRINT: PRINT: PRINT
51260     PRINT TAB(15);"The program is searching for the lightest"
51265     PRINT TAB(15);"W-section that meets all requirements for"
51270     PRINT TAB(15);"member ";I
51275     PRINT: PRINT TAB(20);"one moment please..."
51280     LB=LB(I)/12
51285     L=ELENG(I)/12
51290     A=ELENG(I)/12
51295     VUMAX=VMAX(I)
51300     MUMAX=MMAX(I)/12
51305     MA=MA(I)/12: MB=MB(I)/12
51310     GOSUB 51325: ' DESIGN SUBROUTINE
51315 NEXT I
51320 RETURN
51325 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
51330 IF ABS(MA)=0 AND ABS(MB)=0  THEN 51335 ELSE 51340
51335 CB=1 : GOTO 51415
51340 IF LB < L THEN 51345 ELSE 51390
51345 IF ABS(MA) < ABS(MB) THEN 51350 ELSE 51355
51350 M11=MA : M12=MC : GOTO 51360
51355 M11=MC : M12=MA : GOTO 51360
51360 IF ABS(MC) < ABS(MB) THEN 51365 ELSE 51370
51365 M21=MC : M22=MB : GOTO 51375
51370 M21=MB : M22=MC : GOTO 51375
51375 CB1 = 1.75 + 1.05*(-M11/M12) + .3*(-M11/M12)*(-M11/M12)
51380 CB2 = 1.75 + 1.05*(-M21/M22) + .3*(-M21/M22)*(-M21/M22)
51385 IF CB1 < CB2 THEN CB=CB1 ELSE CB=CB2
51390 IF LB=L THEN 51395 ELSE 51415
51395 IF ABS(MA) < ABS(MB) THEN 51400 ELSE 51405
51400 M1=MA : M2=MB : GOTO 51410
51405 M1=MB : M2=MA : GOTO 51410
51410 CB = 1.75 + 1.05*(-M1/M2) + .3*(-M1/M2)*(-M1/M2)
51415 IF CB > 2.3 THEN CB=2.3
51420 WTMIN=1000
51425 OPEN "BEAM.DAT" FOR INPUT AS #1
51430 IF EOF(1) THEN 51715
51435 INPUT #1,ND,WT,SX,D,TW,BF,TF,RT,DAF,ZX,RY,AG,IY,JT,KF
51440 IF D <= DMAX AND D=> DMIN THEN 51445 ELSE 51430
51445 H = D - TF
51450 IX = SX*(D/2)
51455 CW = H*H*IY/4
51460 AWEB = D*TW
51465 HC = D-2*KF
51470 KV = 5 + 5/((A/HC)*(A/HC))
51475 X1 = (PI/SX)*SQR(E*G*JT*AG/2)
51480 X2 = (4*CW/IY)*(SX/(G*JT))*(SX/(G*JT))
51485 LLTB = LB*12/RY
51490 LPLTB = 300/SQR(FY)
51495 LRLTB = (X1/(FY-10))*SQR(1+SQR(1+X2*(FY-10)*(FY-10)))
```

```
51500 LFLB = BF/(2*TF)
51505 LPFLB = 65/SQR(FY)
51510 LRFLB = 147/SQR(FY-10)
51515 LWLB = HC/TW
51520 LPWLB = 640/SQR(FY)
51525 LRWLB = 970/SQR(FY)
51530 MRLTB = (FY-10)*SX/12
51535 MRFLB = (FY-10)*SX/12
51540 MRWLB = FY*SX/12
51545 MP = ZX*FY/12
51550 IF LLTB <= LPLTB THEN 51555 ELSE 51560
51555 MNLTB = MP
51560 IF LLTB > LPLTB AND LLTB <= LRLTB THEN 51565 ELSE 51570
51565 MNLTB = CB*(MP-(MP-MRLTB)*(LLTB-LPLTB)/(LRLTB-LPLTB))
51570 IF LLTB > LRLTB THEN 51575 ELSE 51580
51575 MNLTB = (SX*CB*X1*SQR(2))/(LLTB*12)*SQR(1+(X1*X1*X2)/(2*LLTB*LLTB))
51580 IF LFLB <= LPFLB THEN 51585 ELSE 51590
51585 MNFLB = MP
51590 IF LFLB > LPFLB AND LFLB <= LRFLB THEN 51595 ELSE 51600
51595 MNFLB = MP-(MP-MRFLB)*(LFLB-LPFLB)/(LRFLB-LPFLB)
51600 IF LFLB > LRFLB THEN 51605 ELSE 51610
51605 MNFLB = SX*20000/(12*LFLB*LFLB)
51610 IF LWLB <= LPWLB THEN 51615  ELSE 51620
51615 MNWLB = MP
51620 IF LWLB > LPWLB AND LWLB <= LRWLB THEN 51625 ELSE 51630
51625 MNWLB = MP-(MP-MRWLB)*(LWLB-LPWLB)/(LRWLB-LPWLB)
51630 IF LWLB > LRWLB THEN 51635 ELSE 51640
51635 PRINT "something is wrong:  lwlb>lrwlb"
51640 MN = MP
51645 IF MNLTB < MN THEN MN=MNLTB
51650 IF MNFLB < MN THEN MN=MNFLB
51655 IF MNWLB < MN THEN MN=MNWLB
51660 IF A/HC > 3 OR  A/HC > (260/(HC/TW))*(260/(HC/TW)) THEN K=5
51665 IF HC/TW <= 187*SQR(KV/FY) THEN 51670 ELSE 51675
51670 VN = .6*FY*AWEB
51675 IF HC/TW > 187*SQR(KV/FY) AND HC/TW <= 234*SQR(KV/FY) THEN 51680
ELSE 51685
51680 VN = .6*FY*AWEB*187*SQR(KV/FY)/(HC/TW)
51685 IF HC/TW > 234*SQR(KV/FY) THEN 51690 ELSE 51695
51690 VN = AWEB*26200*KV/((HC/TW)*(HC/TW))
51695 IF ABS(VUMAX) <= PHIV*VN THEN 51700 ELSE 51430
51700 IF ABS(MUMAX) <= PHIB*MN THEN 51705 ELSE 51430
51705 IF WT < WTMIN THEN 51710 ELSE 51430
51710 WTMIN=WT : NDMIN=ND : PHIMN=PHIB*MN : PHIVN=PHIV*VN :
IXMIN=IX : IYMIN=IY :GOTO 51430
51715 CLOSE
51720 CLS: PRINT: PRINT: PRINT: PRINT
51725 REM¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
51730 REM                    SCREEN OUTPUT
51735 REM
51740 NDMIN(I)=NDMIN : WTMIN(I)=WTMIN : IXMIN(I)=IXMIN
```

```
51745 IYMIN(I)=IYMIN
51750 CLS: PRINT: PRINT: PRINT
51755 PRINT TAB(30); "PROGRAM RESULTS"
51760 PRINT : PRINT : PRINT
51765 PRINT TAB(30); "MEMBER #"; I: PRINT : PRINT
51770 W$="W" : X$="x"
51775 IF WTMIN=1000 THEN 51780 ELSE 51790
51780 PRINT TAB(15); "There is no W-section that meets your requirements"
51785 GOTO 51795
51790 PRINT TAB(30); "Use "; W$; NDMIN; X$; WTMIN
51795 PRINT : PRINT TAB(20); "moment resistance = "; PHIMN; "kip-ft"
51800 PRINT TAB(14); "maximum factored moment = "; MUMAX; "kip-ft"
51805 PRINT TAB(21); "shear resistance = "; PHIVN; "kips"
51810 PRINT TAB(15); "maximum factored shear = "; VUMAX; "kips"
51815 PRINT : PRINT : PRINT : PRINT TAB(25); "Press <ENTER>
to continue"
51820 H$=INKEY$ : IF H$="" THEN 51820
51825 IF ASC(H$) <> 13 THEN 51820
51830 RETURN
51835 '¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
51840 '                      SUBROUTINE VMMAX
51845 '¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
51850 REM MOMENTS AND SHEARS
51855 REM COMPUTE MOMENTS AND SHEARS AT SEVERAL POINTS OF THE BEAM AND
CHOOSE
51860 REM THE MAXIMUM OF EACH.
51865 N=20
51870 FOR J=1 TO N+1
51875     X(J)=(J-1)*(L/N)
51880     M(J)=M1 + X(J)*V1
51885     V(J)=+V1
51890     IF N1=0 THEN 51945
51895     REM************ UNIFORM LOAD SEGMENT **********************
51900     FOR K=1 TO N1
51905         IF X(J) <= A(K) THEN 51940
51910         IF X(J) => B(K) THEN 51930
51915         M(J)=M(J) - W(K)*(X(J)-A(K))*(X(J)-A(K))*.5
51920         V(J)=V(J) - W(K)*(X(J)-A(K))
51925         GOTO 51940
51930         M(J)=M(J) - W(K)*(B(K)-A(K))*(X(J)-(B(K)+A(K))*.5)
51935         V(J)=V(J) - W(K)*(B(K)-A(K))
51940     NEXT K
51945     IF N2=0 THEN 51980
51950     REM************ CONCENTRATED LOADS **********************
51955     FOR K=1 TO N2
51960         IF X(J) <= C(K) THEN 51975
51965         M(J)=M(J) - P(K)*(X(J)-C(K))
51970         V(J)=V(J) - P(K)
51975     NEXT K
51980     IF J=J% THEN MLB=M(J)
51985     IF ABS(M(J)) > MMAX(I) THEN MMAX(I)=ABS(M(J))
```

```
51990     IF ABS(V(J)) > VMAX(I) THEN VMAX(I)=ABS(V(J))
51995 NEXT J
52000 RETURN
      *********************************************************************
      *                                                                   *
      *                         Module DESCOLM                            *
      *                                                                   *
      *********************************************************************

53000  REM                                                      DESCOLM.BAS
53005 REM @@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@
53010 REM @                       DESIGN COLUMNS                           @
53015 REM @@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@@
53020 REM
53025 REM
53030 REM
53035 GOSUB 53080   :' READ COLUMNS SUBROUTINE
53040 GOSUB 53195 : 'READ END FORCES FOR EACH LC
53045 GOSUB 53240  :' ANALYSIS SUBROUTINE
53050 GOSUB 53330  :' DESIGN SUBROUTINE
53055 FOR I=1 TO NE
53060     PRINT W$;NDMIN(I);X$;WTMIN(I),IXMIN(I),IYMIN(I)
53065 NEXT I
53070 GOTO 53040
53075 END
53080  REM-------------------------------------------------------------
53085  REM                      READ COLUMNS
53090  REM-------------------------------------------------------------
53095  REM READ INFORMATION CONTAINED IN DATA FILE "COLUMNF.DAT",WHICH
WILL
53100  REM BE USED IN THE DESIGN OF COLUMNS. READ FOR EACH COLUMN, LENGTH
AND
53105  REM END FORCES FOR EACH LOAD CONDITION. CALCULATE AXIAL FORCE DUE
TO
53110  REM DEAD, LIVE AND WIND LOADS. CALCULATE MAXIMUM REQUIRED FLEXURAL
53115  REM STRENGTH. ENTER CB, K, FY FOR EACH MEMBER. INITIATE CODE(I)=1
TO
53120  REM IDENTIFY COLUMNS.
53125  COLOR 13,4,10
53130  OPTION BASE 1
53135  CLS:KEY OFF
53140 OPEN "COLUMNF.DAT" FOR INPUT AS #1
53145 INPUT #1,NE
53150 DIM ELENG(NE),PD(NE),PL(NE),PW(NE),MUXD3(NE),MUXD6(NE),MUXL3(NE),
MUXL6(NE),MUXW3(NE),MUXW6(NE),MUX3(NE),MUX6(NE),MUX(NE),M1(NE),LB(NE)
53155 DIM CODE(NE),K(NE),MA(NE),MB(NE),MC(NE),PMAX(NE),P1(NE),P2(NE),
P3(NE),P4(NE),P5(NE),NDMIN(NE),WTMIN(NE),IXMIN(NE),IYMIN(NE)
53160 INPUT #1,I,ELENG,LB,K
53165 WHILE I<>0
53170       CODE(I)=2
53175       ELENG(I)=ELENG : LB(I)= LB : K(I)=K
```

```
53180       INPUT #1,I,ELENG,LB,K
53185 WEND
53190 RETURN
53195 IF EOF(1) THEN 53075
53200 INPUT #1,LC
53205 INPUT #1,COL,F1,F2,F3,F4,F5,F6
53210 WHILE COL<>0
53215       I=COL
53220       PMAX(I)=F1 : MA(I)=F3 : MB(I)=-F6
53225       INPUT #1,COL,F1,F2,F3,F4,F5,F6
53230 WEND
53235 RETURN
53240 REM -----------------------------------------------------------
53245 REM |                       ANALYSIS SUBROUTINE                 |
53250 REM -----------------------------------------------------------
53255 REM ANALYSIS SUBROUTINE TO CALCULATE MAXIMUM FACTORED COMPRESSION
53260 REM FORCES FOR EACH COLUMN.
53265 REM
53270 PRINT "MEMBER"; "  MAXIMUM FACTORED COMPRESSION";"  MAXIMUM
FACTORED MOM"
53275 PRINT "------"; "  ----------------------------";"  --------
------------"
53280 PRINT
53285 FOR I=1 TO NE
53290     IF CODE(I) <> 2 THEN 53305
53295     IF ABS(MA(I)) > ABS(MB(I)) THEN MUX(I)=ABS(MA(I)) ELSE
MUX(I)=ABS(MB(I))
53300     PRINT I,PMAX(I); " KIPS",MUX(I); " KIPS-IN"
53305 NEXT I
53310 LOCATE 23 : PRINT TAB(25); "PRESS <ENTER> TO CONTINUE"
53315 H$=INKEY$ : IF H$="" THEN 53315
53320 IF ASC(H$) <> 13 THEN 53315
53325 RETURN
53330 REM------------------------------------------------------------
53335 REM                         DESIGN SUBROUTINE
53340 REM------------------------------------------------------------
53345 REM DESIGN SUBROUTINE TO PICK BEST W-SECTION THAT MEETS ALL
REQUIREMENTS
53350 REM FOR EACH COLUMN.
53355 REM
53360 REM
53365 PI = 3.141592654#
53370 E = 29000
53375 G = 11200
53380 PHIB = .9
53385 PHIC = .85
53390 DMAX = 18
53395 DMIN = 4
53400 FY = 36
53405 FOR I=1 TO NE
53410     IF CODE(I) <> 2 THEN 53475
```

```
53415      CLS: PRINT: PRINT: PRINT
53420      PRINT TAB(15); "The program is searching for the lightest"
53425      PRINT TAB(15); "W-section that meets all requirements for"
53430      PRINT TAB(15); "member ";I
53435      PRINT : PRINT TAB(20); "one moment please..."
53440      LB = LB(I)/12 : KX= K(I):KY=1
53445      L = ELENG(I)/12
53450      A = ELENG(I)/12
53455      PMAX = PMAX(I)
53460      MUX = MUX(I)/12
53465      MA = MA(I)/12 : MB = MB(I)/12 : MC = MC(I)/12
53470      GOSUB 53485 : ' DESIGN SUBROUTINE
53475 NEXT I
53480 RETURN
53485 REM ¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬¬
53490 IF LB < L THEN 53495 ELSE 53540
53495 IF ABS(MA) < ABS(MB) THEN 53500  ELSE  53505
53500 M11=MA : M12=MC : GOTO 53510
53505 M11=MC : M12=MA : GOTO 53510
53510 IF ABS(MC) < ABS(MB) THEN 53515 ELSE 53520
53515 M21=MC : M22=MB : GOTO 53525
53520 M21=MB : M22=MC : GOTO 53525
53525 CB1 = 1.75 + 1.05*(-M11/M12) + .3*(-M11/M12)*(-M11/M12)
53530 CB2 = 1.75 + 1.05*(-M21/M22) + .3*(-M21/M22)*(-M21/M22)
53535 IF CB1 < CB2 THEN CB=CB1 ELSE CB=CB2
53540 IF LB=L THEN 53545 ELSE 53565
53545 IF ABS(MA) < ABS(MB) THEN 53550  ELSE 53555
53550 M1=MA : M2=MB : GOTO 53560
53555 M1=MB : M2=MA : GOTO 53560
53560 CB = 1.75 + 1.05*(-M1/M2) + .3*(-M1/M2)*(-M1/M2)
53565 IF CB > 2.3 THEN CB=2.3
53570 WTMIN = 1000
53575 OPEN "BEAMCO14.DAT" FOR INPUT AS #2
53580 IF EOF(2) THEN 53985
53585 INPUT #2,ND,WT,AG,D,TW,BF,TF,KF,RT,IX,SX,RX,IY,SY,RY,JT,ZX,ZY
53590 IF D<=DMAX AND D=>DMIN THEN 53595 ELSE 53580
53595 HC = D-2*KF
53600 REM ************* determine nominal compressive strength ************
53605 LFLBC = BF/(2*TF)
53610 LRFLBC = 95/SQR(FY)
53615 LWLBC = HC/TW
53620 LRWLBC = 253/SQR(FY)
53625 LAMBX = KX*L*12*SQR(FY/E)/(RX*PI)
53630 LAMBY = KY*L*12*SQR(FY/E)/(RY*PI)
53635 IF LAMBX > LAMBY THEN LAMBDAC=LAMBX ELSE LAMBDAC=LAMBY
53640 IF LAMBDAC<=1.5 THEN 53645 ELSE 53650
53645 FCR = EXP(-.419*LAMBDAC*LAMBDAC)*FY
53650 IF LAMBDAC>1.5 THEN 53655 ELSE 53660
53655 FCR = .877*FY/(LAMBDAC*LAMBDAC)
53660 PN = AG*FCR
53665 REM ******* determine nominal flexural strength about x-axis ****
```

```
53670 H = D - TF
53675 CW = H*H*IY/4
53680 AWEB = D*TW
53685 T = D-2*KF
53690 KV = 5 + 5/((A/HC)*(A/HC))
53695 X1 = (PI/SX)*SQR(E*G*JT*AG/2)
53700 X2 = (4*CW/IY)*(SX/(G*JT))*(SX/(G*JT))
53705 LLTB = LB*12/RY
53710 LPLTB = 300/SQR(FY)
53715 LRLTB = (X1/(FY-10))*SQR(1+SQR(1+X2*(FY-10)*(FY-10)))
53720 LFLB = BF/(2*TF)
53725 LPFLB = 65/SQR(FY-10)
53730 LRFLB = 147/SQR(FY-10)
53735 LWLB = HC/TW
53740 LPWLB = 640/SQR(FY)
53745 LRWLB = 970/SQR(FY)
53750 MRLTB = (FY-10)*SX/12
53755 MRFLB = (FY-10)*SX/12
53760 MRWLB = FY*SX/12
53765 MPX = ZX*FY/12
53770 IF LLTB<=LPLTB THEN 53775 ELSE 53780
53775 MNLTB = MPX
53780 IF LLTB>LPLTB AND LLTB<=LRLTB THEN 53785 ELSE 53790
53785 MNLTB = CB*(MPX-(MPX-MRLTB)*(LLTB-LPLTB)/(LRLTB-LPLTB))
53790 IF LLTB>LRLTB THEN 53795 ELSE 53800
53795 MNLTB = (SX*CB*X1*SQR(2))/(LLTB*12)*SQR(1+(X1*X1*X2)/(2*LLTB*LLTB))
53800 IF LFLB<=LPFLB THEN 53805 ELSE 53810
53805 MNFLB = MPX
53810 IF LFLB>LPFLB AND LFLB<=LRFLB THEN 53815 ELSE 53820
53815 MNFLB = MPX-(MPX-MRFLB)*(LFLB-LPFLB)/(LRFLB-LPFLB)
53820 IF LFLB>LRFLB THEN 53825 ELSE 53830
53825 MNFLB = SX*20000/(12*LFLB*LFLB)
53830 IF LWLB<=LPWLB THEN 53835 ELSE 53840
53835 MNWLB = MPX
53840 IF LWLB>LPWLB AND LWLB<=LRWLB THEN 53845 ELSE 53850
53845 MNWLB = MPX-(MPX-MRWLB)*(LWLB-LPWLB)/(LRWLB-LPWLB)
53850 IF LWLB>LRWLB THEN 53855 ELSE 53860
53855 PRINT "Something is wrong:  lwlb>lrwlb"
53860 MNX = MPX
53865 IF MNLRB<MNX THEN MNX=MNLTB
53870 IF MNFLB<MNX THEN MNX=MNFLB
53875 IF MNWLB<MNX THEN MNX=MNWLB
53880 REM ******* determine nominal flexural strength about y-axis *****
53885 MPY = FY*ZY/12
53890 MRY = FY*SY/12
53895 IF LFLB<=LPFLB THEN 53900 ELSE 53905
53900 MNY = MPY
53905 IF LFLB>LPFLB AND LFLB<=LRFLB THEN 53910 ELSE 53915
53910 MNY = MPY-(MPY-MRY)*((LFLB-LPFLB)/(LRFLB-LPFLB))
53915 IF LFLB>LRFLB THEN 53920 ELSE 53925
53920 MNY = SY*20000/(12*LFLB*LFLB)
```

```
53925 IF MNY>MPY THEN MNY=MPY
53930 REM *********************** interaction equation ********************
53935 IF PMAX/(PHIC*PN)=>.2 THEN 53940 ELSE 53950
53940 INTERACT = PMAX/(PHIC*PN) + (8/9)*(MUX/(PHIB*MNX)+MUY/(PHIB*MNY))
53945 IF INTERACT<=1 THEN 53965 ELSE 53580
53950 IF PMAX/(PHIC*PN)<.2 THEN 53955 ELSE 53960
53955 INTERACT = PMAX/(2*PHIC*PN) + (MUX/(PHIB*MNX)+MUY/(PHIB*MNY))
53960 IF INTERACT <=1 THEN 53965 ELSE 53580
53965 IF WT<WTMIN THEN 53970 ELSE 53580
53970 WTMIN=WT : NDMIN=ND : PHIMNX=PHIB*MNX : PHIMNY=PHIB*MNY
: PHIPN=PHIC*PN
53975 INTERMIN=INTERACT : IXMIN = IX : IYMIN = IY
53980 PHIVN=PHIV*VN : GOTO 53580
53985 CLOSE #2
53990 REM -------------------------------------------------------------
53995 REM |                    screen output subroutine                 |
54000 REM -------------------------------------------------------------
54005 CLS : COLOR 7,0,1
54010 LOCATE 5
54015 NDMIN(I)=NDMIN : WTMIN(I)=WTMIN : IXMIN(I)=IXMIN :
IYMIN(I)=IYMIN
54020 PRINT TAB(20); "PROGRAM RESULTS FOR COLUMN #";I
54025 PRINT : PRINT : PRINT
54030 W$="W" : X$="x"
54035 IF WTMIN = 1000 THEN 54040 ELSE 54050
54040 PRINT TAB(15); "There is no W-section that meets your requirements"
54045 GOTO 54055
54050 PRINT TAB(30); "Use ";W$;NDMIN;X$;WTMIN
54055 PRINT : PRINT TAB(20); "interaction = ";INTERMIN
54060 PRINT TAB(20); "required compressive strength = ";PMAX;"kips"
54065 PRINT TAB(20); "compressive resistance = ";PHIPN;"kips"
54070 PRINT TAB(20); "required flexural strength about x-axis = ";
MUX;"k-ft"
54075 PRINT TAB(20); "flexural resistance about x-axis = ";PHIMNX;"k-ft"
54080 PRINT TAB(20); "required flexural resistance about y-axis = ";
MUY;"k-ft"
54085 PRINT TAB(20); "flexural resistance about y-axis = ";PHIMNY;"k-ft"
54090 LOCATE 23 : PRINT TAB(25); "Press <ENTER> to continue"
54095 H$=INKEY$ : IF H$="" THEN 54095
54100 IF ASC(H$) <> 13 THEN 54095
54105 REM -------------------------------------------------------------
54110 REM |                   printer output subroutine                 |
54115 REM -------------------------------------------------------------
54120 IF Z$="N" OR Z$="n" THEN 54210
54125 IF Z$="Y" OR Z$="n" THEN 54130
54130 CLS : COLOR 7,0,4
54135 LOCATE 5
54140 LPRINT TAB(20); "PROGRAM RESULTS FOR COLUMN #";I
54145 LPRINT : LPRINT : LPRINT
54150 W$="W" : X$="x"
54155 IF WTMIN = 1000 THEN 54160 ELSE 54170
```

```
54160 LPRINT TAB(15); "There is no W-section that meets your requirements"
54165 GOTO 54175
54170 LPRINT TAB(30); "Use ";W$;NDMIN;X$;WTMIN
54175 LPRINT : LPRINT TAB(20); "interaction = ";INTERMIN
54180 LPRINT TAB(20); "required compressive strength = ";PMAX;"kips"
54185 LPRINT TAB(20); "compressive resistance = ";PHIPN;"kips"
54190 LPRINT TAB(20); "required flexural strength about x-axis = ";
MUX;"k-ft"
54195 LPRINT TAB(20); "flexural resistance about x-axis = ";PHIMNX;"k-ft"
54200 LPRINT TAB(20); "required flexural resistance about y-axis = ";
MUY;"k-ft"
54205 LPRINT TAB(20); "flexural resistance about y-axis = ";PHIMNY;"k-ft"
54210 RETURN
```

The vita has been removed from the scanned document