TIGHT BINDING

OPTIMUM DESIGN OF STEEL FRAMES INCLUDING COMPOSITE ACTION OF STEEL BEAMS AND CONCRETE SLABS AT ULTIMATE LOAD

by

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SYNOPSIS

This thesis presents optimization procedures for the design of steel frames including composite steel beams at ultimate load. The study also includes a theoretical investigation of the effects of providing composite action in a building steel frame at ultimate load and its advantage over the corresponding non-composite steel frame based on the optimum solution.

In this study, a theory of plastic collapse is applied to the design of composite frames in a limited way due to the lack of ductility in the reinforced concrete slab. The optimum solution is found by using linear programming techniques, successively, in order to minimize the weight of the composite steel structures. A computer program generates, automatically, the constraints of the feasible collapse modes for the proposed structure and the corresponding objective function, then produces the optimum solution using linear programming algorithm without violating the safety criteria for the composite beams. The proposed procedure appears to be efficiently applicable when an electronic computer is used.

Furthermore, comparisons of the changes in required plastic moment capacities and relative cost on a weight saving basis of a composite steel frame to that of the corresponding non- omposite one are made with several model frames. This investigation indicated that there is a definite advantage of the composite frames over the conventional steel frames from either economy or structural strength-capacity. Since the composite beams have greater shape factors than the normal steel beams, lighter members can be used, and also, it is not necessary to consider the buckling of the compression flange of the steel beams when the composite construction is adopted.

Finally, an alternative optimization procedure, using the nonlinear programming technique, is proposed to eliminate some of difficulties and arbitrary restrictions which are found in the other method. This approach appears to be powerful in handling the nonlinearity of the minimum weight design problems and also efficient in refining the linear optimum solution.

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NOTATION

The following symbols are used in this paper:

a	=	the distance from the top of concrete slab to N. A. of composite section, when N. A. is in slab;
a_{ji}	=	the coefficient of the variable x_i for the j-th constraint of linear programming problem;
$\bar{a}_{ji}$	=	a_{ji}/b_j;
$a_j(X)$	=	the current value of the j-th constraint of nonlinear programming problem;
A_{ji}		$[A_{ji}]$ = a m x n matrix;
A_s	=	the total area of steel section;
b	=	the effective width of concrete slab;
b_j	=	the external work for the j-th collapse mechanism and also the coefficient of the dual variable U_j for the dual objective function G (U) defined by $G(U) = \sum_1^m b_j U_j$;
$\bar{b}_i, \underline{b}_i$	=	upper and lower bounds, respectively, for the variable x_i;
B	=	boundary of the feasible region;
C, C'	=	the compressive forces of concrete section and steel section, respectively;
C_i	=	the coefficient of the primal variable x_i for the primal objective function F(X) defined by $F(X) = \sum_1^n C_i x_i$;
d	=	depth of steel section;
d_{kx}	=	the displacement of the load P_k for the collapse mechanism x;
D	=	a feasible region;
D'	=	a region in E_n which controls D;
e, e', e''	=	moment arms, Eqs. (1. 2) and (1. 3);
E_n	=	Euclidian n-dimensional space;
f	=	Z_p/Z_e = a shape factor;
f_c'	=	the compressive strength of concrete;
F	=	an objective function;
F_{max}	=	the optimum value of the objective function in nonlinear programming problem;

F_w	=	the primal weight function;
$F(X)$	=	$- W(X)$ = the current value of the objective function in nonlinear programming problem; the primal objective function;
F_y	=	the yield strength of steel ; t
$g(X)$	=	$\nabla F(X)$ = the gradient of the objective function;
G	=	the intersection of the hypersurfaces $G_1, \ldots, G_q$;
G_w	=	the dual weight function;
$G(U)$	=	the dual objective function;
$G_j(X)$	=	the hypersurface in E_n defined by $a_j(X) = 0$;
$H_j(X_o)$	=	the supporting hyperplane to G_j at X_o;
I	=	an identity matrix;
L	=	the maximum length of D'; also span length;
L_i	=	the length of the i-th member;
L_s	=	shear span length;
m	=	the number of constraints ; t
M	=	bending moment;
M_a	=	allowable moment;
M_p	=	fully plastic moment capacity; also the fully plastic moment capacity of steel section alone;
$\overline{M}_{p_o}$	=	the starting point to obtain $\overline{M}_{p_{min}}$;
$\overline{M}_{p_{min}}$	=	X_{max} = the nonlinear minimum weight design solution;
M_r	=	redundant moment;
M_u	=	ultimate moment of composite section;
M_x	=	moment anywhere at distance x from the origin of the member;
n	=	the number of variables; also the number of shear connectors within L_s;
N.A.	=	neutral axis;
P	=	service load;
P_u	=	aP = ultimate load;

$P_q(X)$ = $I - U_q(X)\, V_q(X)\, U_q^T(X)$ = the symmetric n x n projection matrix;

q = the current number of active constraints;

q_u = the ultimate strength of one shear connector ;

C = the total horizontal shear force in all shear connectors;

$Q(X_o)$ = the intersection of the supporting hyperplanes $H_1(X_o), \ldots, H_q(X_o)$ at X_o;

$R(X_v)$ = $V_q(X_v\; U_q^T(X_v)\, g$ = a vector of order q;

s = safety factor or load factor;

t = slab thickness; also the step length;

T = the tensile force in steel section;

$u_j(X)$ = $\nabla a_j(X)$ = the gradient of $a_j(X)$, a vector of order n;

U = the distance from the original point to the section of the plastic hinge formed due to the distributed loading;

U_j = the j-th dual variable;

$U_q(X)$ = $[\, u_1(X), \ldots, u_q(X)]$ = a q x q matrix;

v_{ii} = the i-th diagonal element of $V_q(X)$;

w_i = the unit weight of the i-th member;

W = total weight of a frame;

W(X) = the current weight of the frame;

$W_q(X)$ = $[a_1(X), \ldots, a_q(X)]$ = the error vector of order q ;

$|W_q(X)|$ = the length of error vector, a measure of the nearness of the point X_v to the intersection G;

x_i = the current value of the i-th design variable; also the i-th primal variable;

x_v = a point which lies near the intersection, G, of q constraints, $1 \leq q \leq m$;

X = $[x_1, \ldots, x_n]$ = the current design, a vector of order n, a point in E_n;

X_{max} = the optimum solution;

Z_e = elastic section modulus;

Z_p = plastic section modulus;

$Z(X_v)$	=	$P_q(X_v)\,g/\lvert P_q(X_v)\,g\rvert$ = a unit vector of order n which points in the direction of the projected gradient;
α	=	M_u/M_p;
$\beta(X_v)$	=	max $[\,\lvert P_q(X_v)\,q\rvert, \beta_\ell\,]$;
$\beta_i(X_v)$	=	$v_i(X_v)/2 \quad v_{ii}$;
$\beta_\ell(X_v)$	=	max $[\beta_i(X_v)]$;
σ_a	=	the allowable stress in bending;
σ_y	=	the yield strength in bending;
δ	:	a tolerance which controls the nearness of the point X_v to the intersection G;
ϵ	=	a tolerance which controls the nearness of the point X_{max} to the true optimum point;
θ	=	the plastic hinge rotation;
θ_p	=	the plastic hinge rotation capacity;
$\nabla a_j(X)$	=	$u_j(X)$ = the gradient of $a_j(X)$ at X, a vector of order n;
$\nabla F(X)$	=	$g(X)$ = the gradient of the objective function at X, a vector of order n;

CHAPTER I

INTRODUCTION

1.1 Object and Scope of Study

The basic advantages of composite construction compared to non-composite construction are the savings in materials, greater stiffness of a floor system, and also greater overload capacity of a floor. Composite construction, for these reasons, are becoming popular both in bridges and more recently in building frames.

This thesis presents an optimization procedure for the design of steel frames including the composite action of steel beams and concrete slabs at ultimate load. The study also includes a theoretical investigation of the effects of providing composite action in a building steel frame at ultimate load, and its advantages over the corresponding non-composite steel frame based on the optimum solution.

The object of this investigation is to demonstrate the feasibility of using programming techniques for obtaining optimum plastic design of rigid steel frames with composite beams, and also to find the effects of composite action in a building frame, so that the full benefit afforded by composite action is realized.

The nonlinear analysis and design inherent in the plastic design of steel structures or the limit design of concrete structures have the advantage over the methods of analysis based on the normal assumption of elastic structural behavior. The plastic design procedures for steel structures first received extensive acceptance in the AISC Specification of 1961. Limit design for concrete structures has received only limited approval in the ACI Building Code of 1963, due to the uncertain and limited ductility of reinforced concrete members. Both of these nonlinear methods have found acceptance in many countries of Europe and in general throughout the world. However the application of plastic design method to steel frames with composite beams has not found its way into full acceptance up to this point because of the uncertain and limited ductility of reinforced concrete slab. In this study, a theory of plastic collapse is employed in a limited way due to the lack of ductility in the reinforced concrete slab.

The steel sections used in this study will be rolled sections without cover plates. A non-symmetrical section, with cover plates at the bottom flange of the composite section, in combination with cover plates at the negative moment region, is more economical. In general, further reduction in steel weight can be achieved by transforming steel from the top flange to the bottom flange as it is done in built-up sections. However, the prime concern of this study is to determine the effect of composite action based only on the rolled steel section alone, so that fabrication will also be limited to a minimum.

The minimum weight plastic design method is applied to the design of composite and corresponding non-composite frames. In the plastic design of structures, the main difficulty has been in arriving at the correct mode of failure which occurs at the ultimate load. This collapse mechanism, and the ultimate load at which it occurs, are used as the basis of design. An examination of all possible modes of failure normally requires the testing of a very large number of failure mechanisms. For a structure of even moderate complexity this has made the procedure somewhat impractical to date. The computer program that has been developed overcomes these difficulties by generating only the equilibrium equations of the feasible modes. For the optimization of steel frames with composite beams, the successive linear programming techniques are used.

Finally, the effects of nonlinearity in the plastic optimization problems of both non-composite steel frames and composite steel frames are investigated by employing the nonlinear programming techniques as an optimum tool.

The computer used for this study is an IBM System 360/Model 65 computer.

1.2 Organization of Thesis

Chapter 2 illustrates how the optimum design of the steel frames with composite beams can be formulated.

Chapter 3 includes a general discussion and detailed presentation of the linear and nonlinear programming methods, including modifications and revisions which were found to be necessary.

In Chapter 4, the effects of the composite action in the building frames at ultimate load, based on the results of optimum solutions, are investigated. A comparison of the changes in plastic moment capacities and relative cost on a weight saving basis, of a composite steel frame to a corresponding non-composite steel frame, is made, with reference to typical two-story and two-bay rigid frame. (Fig. 4-1)

Several design examples of composite frames, based on the linear programming solutions, are presented in Chapter 5.

In Chapter 6, the nonlinearity in the plastic optimum design problems of both non-composite steel frames and composite steel frames are investigated in a general manner and also the feasibility of using the nonlinear programming techniques are discussed with some examples.

Chapter 7 draws several conclusions, based on the results of this study.

1.3 Review of Plastic Theory for Steel Frames

It is shown here the brief description of structural behavior up to collapse. It is followed with the fundamental theorems which are later applied to the minimum weight design problems of steel frames.

The theory of plastic collapse is applicable to the flexural behavior of a ductile material. Such a moment-rotation relationship for steel as shown in Fig. (1-2) is determined by assuming that each fiber will strain according to the idealized stress-strain curve shown in Fig. (1-1). Fig. (1-2) shows that after the maximum moment is reached, the rotation becomes indefinitely large. This moment, termed the plastic moment M_p is the ultimate moment which this section can maintain under the above assumption.

Upon attainment of the plastic moment at a particular cross section, rotation occurs with no increase in the bending moment, and the plastic hinge is said to have formed there.

Collapse occurs only when a sufficient number of plastic hinges developed to transform part or all of a structure into a mechanism. For structures composed of members with constant sections, the mechanism deformation will develop first at points with peak moment, where plastic hinges form and the intermediate elastic parts do not deform further. Final structural deformation then occurs without change in load.

If the collapse occurs when the frame is subjected to the magnitude of the applied load, it is termed the ultimate load. The ratio between the ultimate load P_u and service load P is referred to as the safety factor or load factor. The structure will not transform into a mechanism under $P < P_u$.

The ultimate load also can be shown to lie between the loads computed by the so-called upper-bound and lower- bound theorems. The methods of analysis in the theory of plastic collapse are based on these theorems.

i) Upper-bound Theorem. A load computed on the basis of an assumed mechanism will always be greater, or at best equal to the true ultimate load.

ii) Lower-bound Theorem. A load computed on the basis of an assumed moment distribution which is in equilibrium with the applied loading and where no moment exceeds Mp is less than, or at best equal to the true ultimate load.

At the point of collapse the following conditions must be met:

1) The applied loads must be in equilibrium with the internal forces.

2) There must be a sufficient number of plastic hinges for the formation of a mechanism.

3) The plastic moment must not be exceeded at any point in the structure.

The mechanism method in which the principle of virtual work is used, is based on the upper-bound theorem. The virtual work principle can be stated as follows.

If a system of forces in equilibrium is subjected to a virtual displacement, the work done by the external forces is equal to the work done by the internal forces.

In the plastic analysis, virtual work is generated by subjecting the frame to virtual displacements consistent with the motion of the mechanism. The external work is equal to the sum of the products of all the forces times the displacements through which they move, while the internal work is equal to the sum of the products of the plastic moments times their hinge rotations. No internal work is performed by the moments between the hinges, since there is no change in the curvature in the elastic parts of the structure.

It is convenient to classify mechanisms as independent or elementary mechanisms and combined mechanisms. Usually the elementary mechanisms are beam mechanisms and sidesway or sway mechanisms. If more than two members meet at a joint, an additional elementary mechanism is the joint mechanism. In addition various combinations of independent mechanisms can be made.

In the above concept in plastic design theory, however, the effects of axial loads and shear forces are not taken into account. It is also assumed that the instability effects can be ignored, and the deflection limitation is secondary. All these concepts and assumptions are used in the following chapters except specially dealt-with cases.

1.4 Review of Plastic Theory for Composite Beams

The plastic design of composite beams presents some interesting problems not found in elastic design as follows:

1) There is no difference between the shored and the unshored composite beams in plastic design.

2) It is not necessary to locate the shear connectors close to the bearings, as long as the same total number of shear connectors required are used.

The summary of plastic theory of the composite beams are presented in the following parts. For a more detailed presentation of the plastic theory including the experimental results, the papers of (1), (2), (3), and (4) should be consulted.

Plastic Behavior of Composite Beams

The concrete slab acts as a cover plate in the composite beam. This is made possible by the use of shear connectors which ensure that the steel beam and concrete act as a unit. If sufficient shear connectors are provided, the full plastic moment may be reached without the failure of the connectors.

When the moment on the positive composite section is increased beyond the working load, the bottom flange of the steel beam yields first and the neutral axis moves upwards causing tensile cracks at the underside of the slab. Cracking or spalling of the concrete at the compression face occurs when the strain reaches about 0.38 percent. The ultimate strength of the section is then almost reached. As the curvature of the section increases, crushing of the slab extends downwards, while strain hardening begins in the bottom flange of the beam and the load being carried remains approximately constant. As the curvature is increased still further, the carrying capacity of the composite section is eventually reduced by extensive crushing of the concrete.

This general pattern of behavior is exhibited by beams subjected to positive bending and for the case where the neutral axis is near the top steel flange or in the slab. In cases where the number of connectors is limited to such an extent, the failure of the connectors occurs before the maximum moment is developed. The ultimate moment capacity may also not develop if the plastic neutral axis is close to the bottom beam flange, as crushing at the surface of the concrete occurs before full plasticity can develop in the steel section.

1) Shored and Unshored Composite Beams

It may be of interest to compare the differences in the behaviors, up to failure, between the shored and the unshored composite beams. Fig. (1-3) and Fig.(1-4) show the differences in the stress distributions and the deflections, respectively between the shored and the unshored composite beams. At the ultimate load, they reach the same stress distribution.

(a) Stress Distributions

In unshored construction, the steel beams have an initial stress distribution, as shown in Fig. (1-3). (stage 1) This is due to the dead load to be carried before composite action takes place. After composite behavior exists, the beam has stress distributions shown in stage 2.

The stress distributions for the shored composite beam are as shown in stage 2, where the dead and live loads are applied after composite action exists.

Although the bottom flanges reach the yield point as the live load increases, a greater load can be carried in both cases, because parts of the cross sections still remain elastic. The plastic behavior proceeds with increased load producing the stress distributions shown in stage 3.

The stress distribution at the ultimate load stage is the same for both the shored and unshored cases, as shown in stage 4. Therefore, the ultimate strength is independent of the loading process. The resisting moment of the cross section is equal to the external moment produced by the total load.

(b) Deflections

Fig. (1-4) shows the change in the deflection, covering stage 1 to stage 4, in accordance with the change in the stress previously noted. From these results, it is noted that the deflections due to live load, after composite action exists, are the same for the shored and the unshored composite beams. There is no difference between the shored and the unshored composite members in so far as the plastic design method is concerned which is based on the failure load. According to the classical method, different sections are designed for the shored and the unshored composite members, respectively, for the same design load. It is based on the working stress design method. Such a design method does not take full advantage of the structural properties.

2) Plastic Rotation Capacities of Composite Beams

When applying the plastic design method to composite beams, it is necessary that the composite beams have sufficient plastic rotation capacities. Although the steel is very ductile, the concrete is not, being relatively brittle. The moment curvature diagrams for the composite beams made from these materials, calculated from the stress-strain diagrams that were obtained from material tests of the concrete and the steel have been presented by Spillers [1]. Fig. (1-6) shows the relationship between moment and curvature for the composite girders having the section shown in Fig. (1-5) for various strength of concrete. In Fig. (1-7) the effect of different widths of the cover plate, when the concrete strength is constant, is shown. If a large steel section in comparision to the concrete section is used, the rotation capacity decreases and the girder may fail due to the crushing of the concrete before the load reaches the ultimate load as calculated using the plastic design theory. Thus, when a larger concrete section having a high strength is used, a greater rotation capacity is obtained. These relationships are similar to those for reinforced concrete between the rotation capacity and the ratio of the area of the reinforcing steel to the area of the concrete.

3) Shape Factor of Composite Beams

The safety factor for the failure of a simply supported beam, designed by the working stress method, is given by

$$s = \frac{M_p}{M_a} = \frac{\sigma_y}{\sigma_a} \quad \frac{Z_p}{Z_e} = \frac{\sigma_y}{\sigma_a} \; f \qquad (1\text{-}1)$$

Where:

M_p	= Fully Plastic Moment	M_a	= Allowable Moment
σ_y	= Yield Strength	σ_a	= Allowable Stress
Z_p	= Plastic Section Modulus	Z_e	= Elastic Section Modulus
f	= Shape Factor = Z_p/Z_e	s	= Safety Factor

The shape factor in Eq. (1-1) depends on the shape of the section. Structures designed to have a constant allowable stress for the same design loads, obtain a bigger safety factor against failure, when a greater shape factor f is used. However, in the plastic design method, structures are designed for a given safety factor at failure, the section with the larger shape factor f is more economical.

The shape factors of shored and unshored composite beams are different. Also the ratio of the load before and after composite action results in different values of the shape factors.

Ultimate Strength of Composite Beams

The concept of ultimate strength design in reinforced concrete can also be applied to the design of a composite concrete and steel beam. This may be classified into two categories: the first is based solely on the statical equilibrium of internal forces and the second takes into account the distribution of strains in addition to equilibrium of internal forces.

The second theory can theoretically account for the effect of strain hardening which can contribute as much as 25% to the carrying capacity of the beam, but it has not become popular as it is difficult to apply.

The first theory is based on the assumption that at the maximum flexural capacity, each element of the section has reached the plastic state of stress. This method has the advantage of simplicity and represents the lower limit of the bending capacity. Experiments conducted by Slutter and Driscoll [2] showed that the theoretical ultimate moment based on this theory can be reached as long as sufficient shear connectors are provided. In this study, the ultimate strength of a composite beam shall be calculated from the fully plastic distribution of stresses.

Normally, a composite beam fails due to the crushing of the concrete slab. At this instant a fully plastic state of stress is assumed for both concrete and steel. The assumed stress distribution is shown in Fig. (1-8). The plastic state of stress is represented by stresses of $0.85\, f_c'$ in concrete and F_y in the steel. Case I applies when the neutral axis is in the slab or when the concrete slab is adequate to resist the total compressive force at ultimate load. Case II applies when the neutral axis is below the slab or the concrete slab is not large enough to resist the total compressive force.

Referring to Fig. (1-8) the appropriate equations for computing the ultimate moments are:

Case I

$$
\begin{aligned}
C &= 0.85\, f_c'\, b a \\
T &= A_s \cdot F_y \\
C &= T \qquad\qquad (1\text{-}2) \\
a &= A_s F_y / (0.85\, f_c'\, b) \\
M_u &= Te = T\,(d/2 + t - a/2)
\end{aligned}
$$

Case II $C = 0.85\, f_c'\, bt$

$$T = A_s \cdot F_y - C'$$

$$T = C + C' \qquad (1\text{-}3)$$

$$M_u = Ce' + C'e''$$

The values of e' and e'' must be determined from the stress distribution and geometry of the cross section.

The ultimate moment for the negative moment regions is determined by a similar consideration of only the steel portion of the cross section. The longitudinal slab reinforcing steel may be considered if shear connectors are provided in the negative moment regions. However, in this study, the ultimate moment is reduced to the plastic moment of the steel member to neglect the effect of slab steel in the negative moment regions.

Ultimate Strength of Stud

The requirement for the shear connectors, when designing composite beams according to the plastic design method, are prescribed as follows:

1) The steel beam and the concrete slab to be connected together so that the composite beam can be subjected to its full ultimate capacity.

2) Shear connectors to be designed not to fail before the composite beam does.

Therefore, it is important to know the ultimate strength properties of the shear connectors in detail before their use in design. The result of the research done on the frequently used shear connectors are shown herein.

Generally, the ultimate strength of these connectors is investigated by means of the push-out test. Many experimental results are reported. These reports term studs to be flexible shear connectors which permit some slip as the force increases to the ultimate load. The typical behavior of studs investigated by Thürlimann [3] is shown in Fig. (1-9).

The Bernouilli-Euler linear strain assumption is not satisfied when slips occur in the composite section. The AASHO Specification [5] defines the ultimate design capacity to be the useful capacity that corresponds to the load producing a slip of 0.003 inch divided by a safety factor.

However, it is reported that slip does not affect greatly the ultimate load-carrying capacity. Hence, the plastic design method can be based on the ultimate load-carrying capacity. The simple beam test is used to investigate the shear connectors under the conditions that are close to their actual behavior. Symmetrical point loads are applied, as shown in Fig. (1-10), to the simply supported beam having shear connectors. The shear connectors are subjected to horizontal shearing forces produced by these loads.

The load-carrying capacity of the shear connector investigated is shown as follows. Assuming that the beam fails when subjected to the ultimate bending moment, the compressive force C in the slab is balanced by Σq_{cr}, the summation of the ultimate loads of the shear connectors within the shear span length Ls.

$$C = \Sigma q_u = n\, q_u \qquad (1\text{-}4)$$

Where, C is obtained from the stress distribution at the ultimate bending moment. The ultimate load capacity for a connector is given by:

$$q_u = C/n \qquad (1\text{-}5)$$

Slutter and Driscoll [2] introduced their experimental formulae for the ultimate loads of shear connectors based on the results of many pushout and beam tests. The AISC Specification [6] established the design strength to be this ultimate load divided by the safety factor of 2.5.

Design of Shear Connectors

Sufficient shear connectors are to be located so that the composite beam, made of concrete slab and steel beam, does not fail prematurely before being subjected to the fully plastic moment. The properties of the shear connectors have already been covered. For convenience, the relationship between load and slip for the shear connectors can be simplified, as shown in Fig. (1-11), and used in the investigation of the behavior of the shear connectors up to the failure. The approach of the plastic design method is tried.

In the case of proportional loading, generally, the horizontal shear force on each connector (connector force) is not the same. With the increase in the forces on the shear connectors some slip occurs, at the most highly stressed connectors, this transfers load to the remaining elastic connectors which finally reach their ultimate loading capacity. When all connectors reach their ultimate strength values, the beam fails by a rupture of the shear connectors.

The total horizontal shear force Q in all shear connectors is, regardless of the spacing of the connectors, given by

$$Q = \Sigma q_u = n q_u \qquad (1\text{-}6)$$

Where n is the number of shear connectors within the shear span and q_u is ultimate load for one shear connector.

If the shear connectors fail when the ultimate load capacity of the composite beam is reached, the shear connectors are ideally designed.

From the equilibrium of the stress distribution in the cross section at the ultimate load:

$$C = T = A_s \cdot F_y \quad \text{(Neutral axis remains within slab)} \tag{1-7}$$

Where A_s is area of the cross section of the steel beam and F_y is the yield stress of the steel.

Letting Q = C in Eqs. (1-6) and (1. 7), the number of required shear connectors is given by:

$$n = \frac{A_s \cdot F_y}{q_u} \tag{1-8}$$

The paper of (2) reported that the composite beams do not fail prematurely when the number of shear connectors used is greater than that calculated by Eq. (1-8). The beams fail at a lower load when the number of shear connectors used is less than that given by Eq. (1-8). The paper by Kurata and Shodo [4] also shows the design of shear connectors in the case of moving loads.

1.5 Previous Applications of Programming Techniques to Optimum Design Problems of Steel Frames

From the view point of structural design, Curbon [8] defines the optimization problem as "the optimization of a structure consists of designing and constructing that structure at the lowest cost, with the object of fulfilling a well defined purpose. In particular, the safety factor of the structure must be specified... In estimating the cost, consideration must, of course, be given to the service life of the structure, its maintenance, and the possibility of its adaptation to meet any foreseeable changes in the service required."

However the term of optimum design of the frame used in this thesis will be confined to the problem of minimum weight design. Although the weight of a structure is not necessarily the best measure of optimality of civil engineering structures, it may nevertheless be one of the considerations in design, and then a saving in weight can be of importance. In this case, it is assumed that the weight of the structure determines the cost. This problem may be stated as follows. For given member length of plane frame, together with a set of static loads acting upon it, select the member sections that will make the weight of steel used minimum without violating the constraints for the safety criteria.

When the objective function considered for optimization and the constraint inequalities or equations are linear combinations of the variables, the problem is one of linear programming. All programming problems that are not linear but satisfy the general definition for an optimization problem given above are called nonlinear.

1) Application of Linear Programming Techniques to Minimum Weight Design Problems

During the last decade, the application of linear programming techniques to minimum weight plastic design problems has been done by several researchers [12, 13, 14].

An early study by Heyman [9] showed that the method of inequalities could readily be adapted to the problem of designing structural frames for minimum weight of steel consumed.

From the theorem of lower bound, the bending moment M_{x_i} anywhere at distance x from the origin of the member i must not exceed the plastic moment capacity M_{p_i} of the member. Expressed mathematically it takes the form

$$M_{p_i} \geq M_{x_i} \geq - M_{p_i} \quad (i = 1, \ldots n). \tag{1-9}$$

The moment equilibrium equation requires that M_{x_i} must be expressed in the form of the external load, sP_k, and a set of independent redundant bending moment, M_r. M_{x_i} is implicit in the statement.

$$M_{x_i} = B_{xik} \cdot sP_k + C_{xir} \cdot M_r \tag{1-10}$$

Where B and C are constants, and so (1-10) yields

$$M_{p_i} \geq (B_{xik} \cdot sP_k + C_{xir} \cdot M_r) \geq - M_{p_i} \tag{1-11}$$

Eliminating M_r by the method due to Dines (1918), (1-11) can be converted into a set of m inequality of the form

$$\sum_{i=1}^{n} C_{ji} \cdot M_{p_i} \geq B_{ik} \cdot sP_k \tag{1-12}$$

Where D is a constant and j = 1, . . . , m.

Symond and Neal [10] proposed that the mechanism method which employs virtual work equations to find the correct mechanisms of collpase. The virtual work equations correspond to the inequalities of the equilibrium equations (1-12). Foulkes [11] later pointed out that these inequalities could readily be derived by considering them in terms of mechanisms. The advantage of this method is that the procedures for eliminating the redundant bending moments are not required, and the inequalities can be handled easily and readily in a systematic way.

Since the mechanism method is based upon the upper bound theorem, the plastic moment capacity M_p assigned in the design of the structure will generally be greater than or possibly equal to the theoretically calculated value.

The following summary of the method of minimum weight design proposed by Foulkes has been represented here.

Assuming that each member has constant weight per unit length throughout its length, the total weight of the frame W is given by

$$W = \sum_{i=1}^{n} w_i . L_i \qquad (1\text{-}13)$$

Where w_i is the i-th member's weight of the section and L_i is its length.

The weight w_i is the function of the fully plastic moment capacity M_{p_i} and it will be written by

$$w_i = f\,(M_{p_i}) \qquad (1\text{-}14)$$

On the assumption that the weight of the section used in appropriate generalization is directly proportional to its plastic moment capacity, Eq.(1-14) may be approximated by

$$w_i \propto M_{p_i} \qquad (1\text{-}15)$$

Then the objective function to be minimized is expressed as a linear function of plastic moment capacities as follows

$$F_w = \sum_{i=1}^{n} M_{p_i} . L_i$$

If a collapse mechanism x for the frame is assumed, the virtual work equation for this mechanism may be written as

$$s_x . P_k . d_{kx} = M_{p_i} . \theta_{ix}$$

Where d_{kx} are the displacements of the loads, and θ_{ix} are the hinge rotations of the members relevant to the mechanism x.

In order that the structure is not to transform into a collapse mechanism x, it is necessary to satisfy the safety criteria, $s_x \geq 1$, i. e.

$$\frac{\theta_{ix}}{P_k \cdot d_{kx}} \quad M_{p_i} \geq 1$$

Letting

$$\bar{a}_{xi} = \frac{\theta_{ix}}{P_k \cdot d_{kx}}$$

This inequality may be rewritten by

$$\bar{a}_{xi} \cdot M_{p_i} \geq 1 \quad (i = 1, \ldots, n) \qquad (1\text{-}16)$$

These inequalities are the same as the set (1-12), which are derived by the inequality method.

Under the above assumption, a problem in plastic minimum weight design method leads to a linear programming problem, which may be stated as follows

$$F_w = \sum_{i=1}^{n} M_{p_i} \cdot L_i \qquad (1\text{-}17)$$

Subject to the m constraint functions

$$\left.\begin{aligned} &\sum_{i=1}^{n} \bar{a}_{ji} \cdot M_{p_i} \geq 1 \quad (i = 1, \ldots, m) \\ &\text{and} \qquad M_{p_i} \geq 0 \quad (i = 1, \ldots, n) \end{aligned}\right\} \qquad (1\text{-}18)$$

Foulkes also established the necessary and sufficient condition for minimum weight design procedure. The Simplex method is a well known technique for the numerical solution of linear programming problems. For a detailed presentation of linear programming techniques used for optimum solution, Section 3.1 shall be referred.

2) Application of Nonlinear Programming Techniques to Minimum Weight Design Problems

It should be pointed out that the applications of the nonlinear programming techniques to the minimum weight design problem in the past developements have invariably been based on the normal assumptions of elastic structural behaviors. Schmit [15] formulated the minimum weight elastic design of a three-bar plane truss as a nonlinear programming problem.

Moses [16] used the cutting plane method which involves solving a succession of linear programming problems to obtain the minimum weight elastic solution to a three bar truss problem and also to obtain the solution to a more practical problem consisting of a single-story single-bay rigid frame.

Using the modified Rosen's Gradient Projection Method of nonlinear programming techniques, Brown and Ang [17] developed the computer optimization method for obtaining minimum weight design of elastic structures.

However, the application of the nonlinear programming techniques to the minimum weight plastic design problems has not found its way into the computer application noted to this point.

The paper by Foulkes [11] investigated the nonlinearity of the plastic moment capacity - unit weight relations. It is concluded that the errors introduced by the linear assumption of these relations are usually small.

The equilibrium method by Douglas [18] showed that the equilibrium equations in which distributed loadings are a factor and the hinge positions are not predetermined, should take the nonlinear form. Such effects of nonlinearity will be investigated in Chapter 6 in a more general manner.

The nonlinear programming problems may be solved by the gradient projection method, the steepest descent method or other similar methods. Nonlinear programming problems can also be transformed into linear programming problems by using a series of linear programming approximations or a series of unconstrained minimizations. For a detailed presentation of nonlinear programming techniques used for optimum solution, Section 3.2 should be referred.

CHAPTER 2

PLASTIC DESIGN OF STEEL BUILDING FRAMES WITH COMPOSITE BEAMS

2.1 General Design Procedures

Plastic composite design has been applied to buildings with success in a limited way. Most of these have been built in England. Johnson, Finlinson and Heyman [19], in the design of a five story building frame, found that plastic composite design gave substantial reduction in cost when compared with the established elastic composite design method. However, the applications of the plastic design method to composite steel frames have been relatively few in number.

In this paper, plastic design of building frames with composite beams is performed in the following way.

1) Determine the dimensions of the structure, working loads and ultimate loads.

2) Obtain the minimum weight design solution for the proposed composite steel structure.

3) Select sections based on the plastic moment capacities of members obtained in step (2).

4) Design the beam-columns.

5) Check the selected sizes for adequacy against local buckling, lateral buckling, excessive deflection, and shear.

6) Design the shear connectors, connections, and lateral bracing systems.

7) Consider the problems of fabrication and erection.

2.2 Assumption on Composite Beam Design

For design purposes, a composite floor is assumed to be a series of T-beams, each made up of one steel beam and a part of the concrete slab.

The width of concrete slab that is assumed effective as the flange of the T-beam is governed by the codes given in the AISC and the CSA Specifications for composite construction [6, 7]. For slabs extending on both sides of a beam, the effective width of the concrete flange must not exceed:

1) one-fourth of the beam span
2) the flange width of the steel beam plus sixteen times the slab thickness.
3) the distance from the center of the steel beam to the centres of adjacent steel beams.

For slabs extending on only one side of a beam, the effective concrete flange width shall not exceed:

1) one-twelfth of the beam span
2) six times the slab thickness
3) one-half of the clear distance to an adjacent steel beam

Furthermore, the ultimate strength is not significantly influenced by shoring, shrinkage or creep, and so except in unusual cases, none of these need be considered in the proportioning of composite beams for buildings.

2.3 Loading Conditions

Normally in building design, the following types of loading are considered; dead loads, live loads, lateral loads and deformational loads such as creep and shrinkage.

The dead loads in building frames will includes the beams, columns, the floor or concrete slab, the walls, partitions and ceiling plasters. The live loads in building design are normally considered as uniformly distributed over the floor area.

At ultimate loads, all dead loads and live loads other than wind or earthquake are carried by the composite section. Opinions differ on the factor by which design loads should be multiplied. The load factor for plastic design of steel is the same for dead load and live load, while in ultimate strength design of concrete structures the dead-load factor is smaller than the live-load factor. A load factor of 1.70 minimum for plastic design of steel is prescribed in the CSA Codes, in which furthermore the reduction in the load factor is permitted for the loading combination of dead load plus live load plus wind load or earthquake load. However, in this study, it is assumed that the overall load factor of 1.70 for all the prescribed design or working loads, though the larger load factor appears better for the design of composite beams.

The selection of members will be based on the minimum weight design solution. In this study, two types of the loading combinations of dead load plus live load and also wind load, were considered into, simultaneously, through the application of the minimum weight design method to the example frames.

It is assumed that these loads are static and that they remain proportional during the entire loading history. Although this assumption is not fulfilled in many cases, study has shown that the usual load fluctuations existing in buildings will not cause a significant reduction of the ultimate load computed by assuming proportional loading. Plastic design should not be used if load fluctuations are severe enough and repeated often enough to cause fatigue failure.

2.4 Application of Plastic Collapse Theory to Steel Frames with Composite Beams

Composite beams are composed of two different materials ; steel possessing ductility and concrete lacking in ductility. Thus the limited ductility of concrete slab makes it difficult to apply the conventional plastic collapse theory to structural design of composite steel frames. On the other hand, the plastic compatibility condition ensures that sections do not fail due to crushing of the concrete slab prior to developing a plastic collapse mechanism. This requires that for any loading level up to collapse the plastic rotation θ_i at the section i should not be larger than the rotation capacity θ_{pi}.

$$\theta_i \leq \theta_{pi} \qquad (2\text{-}1)$$

This condition can be incorporated in the optimization problem as an additional constraint. Alternatively the following approach is taken here.

The previous study [20] on the moment capacities of the composite beam section of a rigid frame at working loads indicates that the first plastic hinges in the composite beams are likely to form at the ends or at the negative moment regions and finally the plastic hinges form at the positive moment regions or near the mid-spans when ultimate loads are reached. Such a tendency is similar to the plastic behavior of the fixed-ended beam in which the first hinge will form at the ends and the hinge at the center will be the last to form; therefore rotation capacity is required at the ends and none is required at the center. Thus, though in a limited way, the plastic collapse theory may be applicable to the composite steel frames under the following conditions.

The plastic moment capacity at the negative moment regions shall be reduced to the plastic moment capacity of the steel section alone in order to allow large rotational deformations, and it gives conservative results. However, the plastic moment capacity at the positive moment region is assumed to be equal to the ultimate moment capacity of the composite beam (see Sec. 4.2), since the positive hinge likely to form last requires a very small rotation capacity. Furthermore, in order to give a sufficient amount of rotation capacity at the positive hinges against the critical condition and also make the foregoing assumptions more realistic, the following points shall be considered.

1) A use of larger concrete section having a high compressive strength gives a greater rotation capacity. (See Sec. 1.4)

2) Limit the maximum value of the ratio of the steel beam section to the effective concrete slab section not to cause the failure of composite sections due to crushing of the concrete slab before the ultimate load is reached. (See Sec. 1.4)

3) Test the plastic compatibility condition given by Eq. (2-1) after the optimization procedure.

2.5 Linear Optimization Procedures

1) Outlines of Optimization Procedures

In this section, we will examine how to develope an efficient computer program which employs linear programming techniques as a tool for optimization of steel frames with composite beams.

The formulation of minimum weight design problem consists of the following steps:

a) Generate all feasible constraints from the elementary mechanisms for the steel frames alone in which the effects of composite action are neglected.

b) Apply linear programming techniques to obtain the minimum weight design solution for the steel frames alone.

c) For each composite beam section at which a positive plastic hinge is likely to form, assume the ratio of the ultimate moment capacity of composite section, M_u, to the plastic moment capacity of the corresponding steel section, M_p, where both M_u and M_p are unknowns, by substituting the value of the plastic moment capacity of the corresponding steel beam alone obtained previously, into M_u/M_p function which are approximated in terms of M_p under the given properties of structural members. (See Sec. 4.2).

d) Modify the elementary mechanisms to assume the variable M_u approximately equal to the variable M_p multiplied by the M_u/M_p ratio which are previously calculated in step (c).

e) Generate all feasible linear constraints from the newly generated elementary mechanisms in step (d) where the effects of composite action between steel beams and concrete slab are taken into account.

f) Seek the linear programming solution for new constraints.

g) Check the convergence of the solution for the composite sections. If all the solutions are satisfactory, then the correct minimum weight design solution is reached. If not, go back to step (c) and repeat the same procedures until the correct minimum weight design solution is found. (See Section 4-2).

These steps are briefly depicted in Fig. (2-1), and the corresponding optimization procedures will be carried out by using computer programs which are composed of a main program and a subroutine. The function of the main program is to generate the equilibrium equations of the feasible modes, to set the linear weight function in terms of variables M_{p_i}; to control the whole procedures until the convergence in the solution reached, and also to read in the input data and print out the minimum weight solution. The function of the subroutine program is to carry out the optimization procedure using the Simplex method in the linear programming. (See Sec. 3. 1).

The input to the computer program consists of a description of the elementary failure mechanisms. These characterize the structure for a particular loading, and suitably combined they result in the feasible collapse mechanisms at the ultimate load. The coefficients of the objective function, which is to be minimized, are also required as input.

The programs were written in FORTRAN with desired flexibility. Whole procedures can be done in the computer cores without using any disc or tape units. The program is constructed to be used by other computer with minimum alternation.

A description of input data, a general flow chart for the main program and computer results for the example problems are presented in Appendix 4.

2) Generation of Equilibrium Equations of Feasible Collapse Mechanisms

In the computer program, the equilibrium equations of the collapse mechanisms are expressed in matrix form. This matrix is termed here a hinge rotation matrix, THETA, which is related to column vector of external work, W_{ext}. This will be written by

$$\sum_{i=1}^{NOMP} \sum_{k=1}^{NTHL} \text{THETA}(j,\ k,\ i) \cdot M_p(i) \geq W_{ext}(j) \qquad (j = 1, 2 \ldots, \text{NELM}) \qquad \ldots\ldots (2\text{-}2)$$

where: NOMP = The number of plastic moments.
NTHL = The number of hinge locations.
NELM = The number of elementary mechanisms.

The outlines of computer procedure for generating the feasible mechanisms are shown in Fig. (2-2), which will be explained as follows.

Generate all feasible combinations of each elementary mechanisms with a set of combined mechanisms. Such work will be done by checking hinge rotation matrix, and accompanied by series of elimination procedure so as to result in only the feasible combined mechanisms. After this cycle, a set of the combined mechanism No. (i + 1) results in. Repeat the similar procedures until no combined mechanisms can be generated. The constraints to be used for linear programming solution are stored in the matrix form which may be termed in this paper as constraint matrix W_{int} given by

$$\sum_{i=1}^{NOMP} W_{int}(j, i) . M_p(i) \geq W_{ext}(j) \qquad (j = 1, 2, \ldots, NTMC) \qquad \ldots\ldots\ldots\ldots\ldots (2\text{-}3)$$

where NTMC is the number of all feasible mechanisms.

2.6 Secondary Design Consideration

With the information from the minimum weight solution for the proposed structure, the selection of member sizes and details can begin. However in addition to furnishing the required plastic modulus, each member must meet certain additional requirements which assure its capacity to maintain the plastic moment through large enough rotations to permit all the hinges in the failure mechanism to form. These secondary checks are shown by:

1) Adjustment of member size for axial force.

2) Design of the shear connectors, connections and lateral bracing system.

3) Local buckling

4) Shear force

5) Frame stability

6) Rotation capacity and deflection

CHAPTER 3
MATHEMATICAL PROGRAMMING TECHNIQUES

3.1 Linear Programming Techniques

1) Formulation of Linear Programming Problem

When the optimization problems of structural design are approximated by the linear objective function and the linear constraints, linear programming techniques are used.

In recent years, the availability of computers and computer programs has made the linear programming method applicable to the problems in which large numbers of variables are involved.

A summary of the linear programming problem and some techniques (21, 22) will be presented before its application to the optimization of the structure is discussed. (See Chapters 4 and 5).

The general problem in linear programming is as follows:

To minimize the linear objective function

$$F = \sum_{i=1}^{n} C_i x_i \tag{3-1}$$

where $x_1, \ldots, x_n$ are the variables of the problem, which are defined by the nature of the problem.

Subject to the m's constraint functions

$$x_i \geq 0 \quad (i = 1, \ldots, n)$$

and

$$\sum_{i=1}^{n} a_{ji} x_i \geq b_j \quad (j = 1, \ldots, m) \tag{3-2}$$

One of the special features of all linear programming problems is the discrepancy between the number of constraints and the number of variables.

As the size of a linear programming problem that can be solved on a electronic computer is usually limited by the number of rows m involved, a problem that is too large when stated in terms of the primal may be of the right dimensions when written as the corresponding dual problem.

From the fundamental theorem of duality,

Primal
minimize

$$F(x_1, \ldots, x_n) = \sum_{i=1}^{n} C_i x_i$$

subject to

$$x_i \geq 0 \quad (i = 1, \ldots, n)$$

$$\sum_{i=1}^{n} a_{ji} x_i \geq b_j \quad (j = 1, \ldots, m)$$

Dual
maximize

$$G(U_1, \ldots, U_m) = \sum_{j=1}^{m} U_j \cdot b_j$$

subject to

$$U_j \geq 0 \quad (j = 1, \ldots, m)$$

$$\sum_{j=1}^{m} a_{ij} U_j \leq C_i \quad (i = 1, \ldots, n)$$

. . . . (3-3)

In the context of Eqs. (3-1) through (3-3), the minimum weight design problem is as follows:

The variables considered herein are assumed to be the fully plastic moment capacities of the structural members. If the weight of the sections are proportional to their fully plastic moment capacities, the objective weight function can be represented by

$$F_w = \sum_{i=1}^{n} M_{p_i} L_i \qquad (3\text{-}4)$$

Where M_{p_i} is the plastic moment of the i-th member, and L_i is its length.

Introducing the column vector (C_i) whose coefficients are the lengths of the members, L_i, the objective function can be stated as follows.

$$(C_i)^T (M_{p_i}) - F_w = 0 \tag{3-4}$$

The constraint inequalities are derived from the theory of plastic collapse with virtual work approach as set forth in Section 2.4

The complete system of linear inequality constraints may be written in matrix form,

$$\begin{aligned} \left[A_{ji}\right] (M_{p_i}) &\geq (b_j) \\ \left[I_{ii}\right] (M_{p_i}) &\geq (o) \end{aligned} \tag{3-5}$$

where; $\left[A_{ji}\right]$ is a matrix of coefficients of M_{pj} terms. (m x n)

(b_j) is a column vector composed of the external work terms of the equilibrium equations. (m x 1)

$\left[I_{ii}\right]$ is an identity matrix, whose diagonal elements are one with all other elements zero. (n x n)

2) Simplex Method and Computational Aspects of Linear Programming Problems

The solution for the specific problem in which there are two variables, can be readily solved by using the graphical method. When the number of constraints and variables are large, this method become useless. In such cases, the Simplex method is used.

Taking the direct expression as a minimization problem, the objective function by Eq. (3-4)' and the constraints by Eq. (3-5) can be arranged in the Simplex Tableau:

	1 … n	(n + 1)
1 … m	A_{ji} (m x n)	$-b_j$ (m x 1)
(m + n)	I_{ii} (n x n)	0 (n x 1)
(m + n + 1)	C_i (1 x n)	0 (1 x 1)

→ minimize F_w (3-6)

Primal Initial Tableau

where i = 1, 2, ..., n and j = 1, 2, ..., m, with m > n

In the minimum weight design problem, the number of constraints usually far exceeds the number of variables. Using the theorem of duality shown in (3.3), the minimization problem will be converted into the maximization problem in order to obtain better solution by computer. Hence the Simplex Tableau in (3.6) will be rearranged as follows:

	1 … m	(m + n)	(m + n + 1)
n	A_{ij} (n x m)	I_{ii} (n x n)	C_i (n x 1)
(n + 1)	$-b_j$ (1 x n)	0 (1 x n)	0 (1 x 1)

→ maximize G_w (3-7)

Dual Initial Tableau

Once initial computational tableau has been constructed, the Simplex procedure calls for the successive application of:

a) The testing of the elements of the last vector to determine whether a maximum solution has been found, i.e., whether all elements of last row vector are less than zero, or at best, equal to zero.

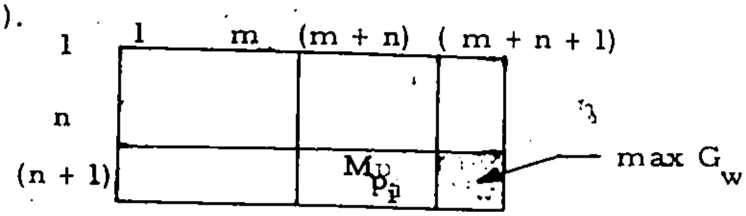
b) The selection of the column k to be introduced into the basis if some elements of last row vector are positive, i. e., selection of the column k with maximum element of last row vector.

c) Test every entry in the pivot column k to see if it is positive or not. If all the entries in the pivotal column are zero or negative numbers, i. e., $a_{ik} = 0$, then the solution is unbounded. If not, find the pivotal element, i. e., min (C_i/a_{ik}).

d) The transformation of the tableau by the complete elimination procedure to obtain the new solution and associated elements.

Each such iteration produces a new basic feasible solution. Eventually a minimum solution will be obtained or a bounded solution determined.

An application of the Simplex procedure to the dual initial tableau in (3-7) yields the transformed values of tableau in (3-8).



(3-8)

From the previous discussion in this section, the maximization of G_w corresponds to the minimization of the F_w. Thus the optimum solution is reached.

$$\min F_w = \max G_w \tag{3-9}$$

Of the several computational method available, Dantzig's Simplex method [23] is used.

3.2 Nonlinear Programming Techniques

1) Formulation of Nonlinear Programming Problem

When the problem of optimum design can be formulated as nonlinear combinations of variables, nonlinear programming techniques can be used, and the related computational algorithm, provide feasible numerical techniques for the determination of an optimum design.

A general problem in nonlinear programming may be defined as follows:

To maximize the objective function:

$$F(X) = F(x_1, x_2, \ldots, x_n) \qquad (3\text{-}10)$$

Where $x_1, x_2, \ldots, x_n$ are the variables of the problem, which are defined by the nature of the problem.

Subject to m constraint functions,

$$a_j(X) = a_j(x_1, x_2, \ldots, x_n) \geq 0$$
$$\text{where } j = 1, \ldots, m, \text{ with } m \geq n \qquad (3\text{-}11)$$

The problem of maximizing F under these conditions is the same as the problem of minimizing -F under the same conditions. In the above formulae, the function F (X) and a_j (X) are assumed to be continuous functions of the variables X. The optimum design problem of structures may be stated as follows: The objective function F is considered as the weight of a structure, but as a programming problem must be expressed in a negative value, and the variables are assumed to be the fully plastic moment capacities of the structural members.

2) Gradient Projection Method and Computational Aspects of Programming Problem

Several methods have been proposed for the solution of nonlinear programming problems. Among the available methods the gradient technique seems to have the greatest flexibility. In particular the "gradient projection method" of Rosen has been found to be well suited to the requirements of structural optimization.

The following summary of the gradient projection method of nonlinear programming has been condensed from the papers of Rosen[24, 25].

An Euclidian n-dimensional vector space E_n is considered. A point in E_n is represented by the vector.

$$X = (x_1, x_2, \ldots, x_n) \qquad (3\text{-}12)$$

A region D within E_n is defined by a set of m constraints in the form

$$a_j(X) \geq 0 \quad (j = 1, 2, \ldots, m) \qquad (3\text{-}13)$$

Corresponding to each function $a_j(X)$ is a hypersurface $G_j(X)$ defined by

$$G_j(X) = [\, X \mid a_j(X) = 0, \; j = 1, 2, \ldots, m] \qquad (3\text{-}14)$$

If a point lies in all of the hypersurface G_j, $j = 1, \ldots, q$, it is said to lie in the intersection G of the q-hypersurfaces. The feasible region D and its boundary B are defined as follows:

$$D = [\, X \mid a_j(X) \geq 0 \,, \; j = 1, 2, \ldots, m] \qquad (3\text{-}15)$$

$$B = [\, X \mid X \text{ in } D, \; a_j(X) = 0 \text{ for at least one } j] \qquad (3\text{-}16)$$

Every point in D is called a feasible point. An interior point X is one for which $a_j(X) > 0$, for all j. The point X is said to lie in a δ-neighborhood of the intersection G if $\Sigma_{j=1}^{q} a_j^2(X) \leq \delta^2$. The region D can be enclosed in a region D' determined by an upper and lower bound on each variable,

$$D' = [X \mid -\underline{b}_i \leq x_i \leq \overline{b}_i, \ i = 1, 2, \ldots, n] \qquad (3\text{-}17)$$

The maximum length L of the region D' is defined by

$$L^2 = \sum_{i=1}^{n} (\underline{b}_i + \overline{b}_i)^2 \qquad (3\text{-}18)$$

At each point X in D' the gradient of each function $a_j(X)$ is a vector

$$u_j(X) = \nabla a_j(X) = \left[\frac{\partial a_j(X)}{\partial x_1}, \ldots, \frac{\partial a_j(X)}{\partial x_n} \right] \qquad (3\text{-}19)$$

Let X_o be a joint which lies in a δ-neighborhood of the intersection G of q-hypersurface (G_j, j = 1, ..., q) where $1 \leq q \leq m$. It is assumed that $u_j(X_o) \neq 0$ for all j and that the q-vectors $u_j(X_o)$ are linearly independent. The vector $u_j(X_o)$ points into D if X is in B and toward D if X is outside of D. The vector $u_j(X_o)$ is orthogonal to the hypersurface G_j at X_o. A hyperplane containing X_o which is orthogonal to $u_j(X_o)$ is given by

$$H_j(X_o) = [X \mid (X^T - X_o^T)\, u_j(X_o) = 0] \qquad (3\text{-}20)$$

$H_j(X_o)$ is the supporting hyperplane to G_j at X_o. Suppose that a point X_o lies in the intersection G of q-hypersurfaces. Corresponding to each hypersurface is a supporting hyperplane $H_j(X_o)$ at X_o. These q-supporting hyperplanes form a linear intersection Q (X_o) which contains X_o.

To compute the vectors $u_j(X_o)$ requires repeated calculation of the first partial derivatives of the constraint functions. This computation can be done conveniently by a forward difference technique. The objective function F is defined for every X in D. The gradient of the objective function is given by

$$g(X) = \left[\frac{\partial F(X)}{\partial x_1}, \dots, \frac{\partial F(X)}{\partial x_n} \right] \tag{3-21}$$

The nonlinear programming problem is then to

$$\text{maximize } F(X) \text{ for } X \text{ in } D. \tag{3-22}$$

A matrix $U_q(X)$ is defined by the normals $u_j(X)$ to $H_j(X)$, where $j = 1, \dots, q \leq m$,

$$U_q(X) = [u_1(X), \dots, u_q(X)], \ (X \text{ in } D) \tag{3-23}$$

The q-by-q symmetrical matrix $U_q^T(X)\, U_q(X)$ is nonsingular for all X in B. Let

$$V_q(X) = [U_q^T(X)\, U_q(X)]^{-1} \tag{3-24}$$

A symmetric n-by-n projection matrix $P_q(X)$ is defined for each X in B by

$$P_q(X) = I - U_q(X)\, V_q(X)\, U_q^T(X) \tag{3-25}$$

In the course of a typical gradient projection calculation it is necessary to obtain the projection of the gradient vector q on an intersection Q . For the purposes of this study $V_q(X)$ is determined by direct inversion.

The stepwise optimization procedure can be described as follows. It is assumed that a feasible starting point is known. If the starting point is an interior point of D the constant gradient g is followed until a boundary point on B is reached. At a boundary point the supporting hyperplanes to the constraint hypersurfaces in which the point lies are determined. The gradient is projected on the intersection of these supporting hyperplanes. A step is taken in the direction of the projected gradient to a new point with an increased value of F. Since the constraints are nonlinear, the new point will generally not be a feasible point. It is then necessary to correct back to the feasible region in such a way that F remains greater than its value prior to taking the step. This correction procedure makes use of the inverse matrix V_q and is intended to determine a corresponding point on G. One measure of the distance of a point X from G is given by the length of the error vector.

$$| W_q (X) | = | [a_1 (X), \ldots, a_q (X)] | \qquad (3\text{-}26)$$

Consider a point X_v and the corresponding matrices $U_q (X_v)$, $V_q (X_v)$ and $P_q (X_v)$ as given by Eqs. (3-23), (3-24) and (3-25).

A q-dimensional vector with components $\gamma_j (X_v)$ is defined by

$$R (X_v) = [\gamma_1 (X_v), \ldots, \gamma_q (X_v)] = V_q (X_v) U_q^T (X_v) q \quad \ldots\ldots \qquad (3\text{-}27)$$

Let

$$\beta_j (X_v) = \gamma_j (X_v)/2 \sqrt{v_{jj}} \qquad (3\text{-}28)$$

where v_{jj} are the diagonal elements of $V_q (X_v)$. Let

$$\beta_\ell = \max_j [\beta_j (X_v)] \qquad (3\text{-}29)$$

$$\beta (X_v) = \max [| P_q (X_v) q | , \beta_\ell (X_v)] \qquad (3\text{-}30)$$

The unit vector $Z(X_v)$ is defined by

$$Z(X_v) = P_q(X_v)\, g / |P_q(X_v)\, g|. \qquad (3\text{-}31)$$

The necessary and sufficient conditions that X_v yield a local optimum are $P_q(X_v)\, g = 0$ and $\gamma_j(X_v) \leq 0$, where $j = 1, 2, \ldots, q$. This means that the gradient must be orthogonal to the intersection G, and the component of the gradient in the direction of the normal to any one of the active constraint hypersurfaces G_j must point away from the feasible region. This is illustrated for the two-dimensional case in Fig. (3-1) In both Fig. (3-1a) and Fig. (3-1b) G is a vertex of R and thus $P_q\, g = 0$. In Fig. (3-1a) X_v is a local optimum because it is not possible to move along either of the curve, $a_1 = 0$ or $a_2 = 0$, such that the objective function F is improved. But in Fig. (3-1b) it is possible to move along the curve $a_1 = 0$ and thus improve F.

Hence, whenever $|P_q\, g| > 0$, it is possible to take a small step of length t in the direction of the projected gradient such that the objective function is improved. This is accomplished by computing the sequence of points.

$$X^{(o)}(t) = X_v + t\, Z(X_v) \qquad (3\text{-}32)$$

$$X^{(j+1)}(t) = X^{(j)}(t) - U_q(X_v)\, V_q(X_v)\, W_q[X^{(j)}(t)]$$

$$\text{where } j = 0, 1, 2 \ldots\ldots. \qquad (3\text{-}33)$$

Eq. (3-32) represents movement along the linear intersection $Q(X_v)$, and Eq. (3-33) represents iteration to correct back to feasible region. (See Fig. (3-2)). In taking the step represented by Eq. (3-32), the constant step length $t = L/100$ in which L is given by Eq. (3-18), was used for the examples presented herein. This value of t was selected after other values led to greater computation times for a typical problem. No attempt has been made to derive an optimum value for t. Using this constant value for t, it is possible to overshoot the optimum point X_{max}; this is illustrated in Fig. (3-3). Whenever this occurs, as indicated by a decrease in the objective function after applying Eqs. (3-32) and (3-33), a local optimum can be located by interpolating along the intersection G.

Whenever $| P_q g | = 0$ but $\gamma_j > 0$ for some j, j = 1, 2, . . . , q, i. e., for $\beta = \beta_\ell > | P_q g |$, the objective function can be improved by dropping the constraint for which $\gamma_j > 0$ and moving along the intersection of the remaining constraints. (see Fig. (3-4)). This is accomplished by using Eqs. (3-32) and (3-33) with P_{q-1}, U_{q-1}, V_{q-1} and W_{q-1} repeating the corresponding quantities in Eq. (3-33). The values P_{q-1}, U_{q-1}, V_{q-1} and W_{q-1} are obtained by omitting u_ℓ and a_ℓ from U_q and W_q, respectively.

It has been shown that a local optimum is also the global optimum if the feasible region D is convex. However, the feasible region for a structural optimization problem is not necessarily convex;for convex D, the gradient projection method may converge to a local optimum which is not global. This difficulty can be overcome, at least intuitively, by starting the algorithm at substantially different points and comparing the resulting solutions.

A tolerance δ is provided which measures the nearness of each constrained point X_v to the corresponding intersection G, and a tolerance ϵ is provided which measures the nearness of the indicated solution to the true optimum. It can be seen that δ and ϵ are not independent. The further a constrained optimum point is from the active constraints, the larger must be the error in the indicated optimum value of the objective function.

A point X_v is said to be on the intersection G if $|W_q(X_v)| \leq \delta$, where $W_q(X_v)$ is defined by Eq. (3-26). The tolerance δ is limited by the accuracy to which the constraint functions can be computed.

Finally, it is assumed that the feasible region D contains points X for which $x_i > 0$, where $i = 1, 2, \ldots, n$. In the course of the calculations one or more variables may approach zero. This can be prevented, if necessary, by defining n constraints in the form of lower founds on each variable.

On the basis of the foregoing adaptation of the basic formulation of the gradient projection method of nonlinear programming, the following algorithm is presented. It is assumed that tolerances $\epsilon > 0$ and $\delta > 0$ have been specified, and that an initial feasible point is known or can be determined.

a) If X_o is an interior point of D, let $X_1 = X_o + t\,g$, where t is chosen so that X_1 is in B. This is a one-dimensional interpolation to determine t such that $a_j(X_o + t\,g) \geq 0$, $j = 1, 2, \ldots, m$, where the equality holds for at least one j. (See Fig. (3-5)).

b) Consider a point X_v which lies in a δ-neighborhood of the intersection G of q constraints, $1 \leq q \leq m$. For convenience let these be G_j, $j = 1, 2, \ldots, q$, so that $|W_q(X_v)| \leq \delta$, where $W_q(X_v)$ is given by Eq. (3-26). Compute $V_q(X_v) = [U_q^T(X_v)\, U_q(X_v)]^{-1}$, $R(X_v) = V_q(X_v)\, U_q^T(X_v)\, g$, $P_q(X_v)\, g = g - U_q(X_v)\, R(X_v)$, and $\beta = \beta(X_v)$ as given by Eq. (3-30). If $\beta \leq \epsilon / 2\, n\, L$, then $X_{max} = X_v$, or if $F(X_v) \leq F(X_{v-1})$, then X_{max} lies along G between X_v and X_{v-1}.

c) If $\beta > \varepsilon/2nL$ and $|P_q g| \geq \beta_\ell$, compute $Z(X_v)$ by Eq. (3-31). (See Fig. (3-2)). A sequence of n points is computed by Eq. (3-32) and (3-33) with $t = L/100$, and n is the smallest integer such that $|W_q[X^{(n)}(t)]| \leq \delta$. If $a_j[X^{(n)}(t)] \geq 0$, $j = q+1, \ldots, m$, let $X_{v+1} = X^{(n)}(t)$. The point X_{v+1} lies in a δ-neighborhood of G. If $a_j[X^{(n)}(t)] < 0$ for at least one j, $j = q+1, \ldots, m$, interpolate for the value of $t' < L/100$ such that $a_j[X^{(n)}(t')] \geq 0$, $j = q+1, \ldots, m$, with the equality holding for at least one j. (See Fig. (3-6)). Let $X_{v+1} = X^{(n)}(t')$. The point X_{v+1} lies in a δ-neighborhood of a new intersection consisting of G and at least one additional constraint.

d) If $\beta > \varepsilon/2nL$ and $|P_q g| < \beta_\ell$ drop u_ℓ from U_q and obtain U_{q-1} and $V_{q-1} = [U_{q-1}^T U_{q-1}]^{-1}$. Compute $R = V_{q-1} U_{q-1}^T g$, $P_{q-1} g = g - U_{q-1} R$, and $Z = P_{q-1} g / |P_{q-1} g|$. A sequence of n points is then computed by Eqs. (3-32) and (3-33) with U_{q-1}, V_{q-1} and W_{q-1} replacing to corresponding quantities U_q, etc. The value $t = L/100$ is used, and n is the smallest integer such that $|W_{q-1}[X^{(n)}(t)]| < \delta$. (See Fig. (3-4)). If $a_j[X^{(n)}(t)] \geq 0$, $j = q+1, \ldots, m$, let $X_{v+1} = X^{(n)}(t)$. The point X_{v+1} lies in a δ-neighborhood of the intersection of the $q-1$ constraints G_j, $j = 1, \ldots, q$, $j \neq \ell$. If $a_j[X^{(n)}(t)] < 0$, for at least one j, $j = q-1, 2, \ldots, m$, interpolate for t' such that $a_j[X^{(n)}(t')] \geq 0$, $j = q+1, \ldots, m$, with the equality holding for at least one j. Let $X_{v+1} = X^{(n)}(t')$. The point X_{v+1} lies in a δ-neighborhood of the intersection of the $q-1$ constraints G_j, $j = 1, \ldots, q$, $j \neq \ell$, and at least one other constraint.

As expected, the determination of the solution to a nonlinear programming problem by the gradient projection method is much more complicated than that of a similar linear programming problem. The cutting plane method of nonlinear programming [26] has been proposed as a simpler method of solution. In this latter method a series of linear programming problems are solved to determine the solution to the original nonlinear problem.

A number of advantages of the gradient projection method over the cutting plane method can be noted. With gradient projection, there is no increase in the number of constraints as the solution proceeds, and it is necessary to linearize only those constraints which are active at a particular stage of calculation. In addition, if the optimum solution does not occur at a vertex of the feasible region, the cutting plane method is even less satisfactory. The capabilities of the basic projected gradient approach for problems of structural optimization appear to be extensive.

CHAPTER 4

INVESTIGATION OF EFFECTS OF COMPOSITE ACTION IN BUILDING FRAME AT ULTIMATE LOAD BASED ON RESULTS OF OPTIMUM DESIGN SOLUTION

4.1 General Remarks

Generally the type of construction controls the economy of a structure and the method of analysis. For structural steel frames two framing schemes are available.

The first type is usually referred to as simple framing. With this type of framing, the end moments are eliminated but it also results in the use of the largest possible beam or girder. However, elimination of the end moments makes possible the use of the simplest connections. There is also no problem as far as analysis is concerned, as only the simple beam moment is required.

The other type is rigid framing. The effect of rigid frame construction is to reduce the size of the steel beams or girders, but the cost of fabricating connections capable of fully rigid action offset the savings in girder material.

For both simple and rigid framing, a more economical design might be obtained by using composite action in the positive moment regions of the beams by providing shear connectors between concrete and the steel sections. In the following sections, we investigate the composite action for rigid framing.

4.2 Empirical Relationship between Ultimate Moment Capacity of Rolled Wide Flanges and That of Corresponding Composite Beams

As noted in Sec. 2.4, the ultimate moment capacity of a composite beam in the positive moment region, M_u, shall be calculated as a function of the structural properties of the corresponding steel wide flange section and concrete slab. In order to make the conventional minimum weight plastic design method applicable to the design of composite steel frames, the ultimate moment capacity of a composite beam in the positive moment region, M_u, shall be approximated as a function of the plastic moment capacity of a rolled wide flange, M_p, assuming that all other factors related to the ultimate moment capacity of the composite beam, i.e., the thickness and effective width of concrete slab and strengths of materials are known. It will be written by

$$M_u = f\,(M_p) \tag{4-1}$$

Since the linear programming techniques are used as an optimum tool, it seems to be convenient to assume that the ultimate moment capacity of a composite beam section, M_u, is directly proportional to a plastic moment capacity of the wide flange, M_p, as given by

$$M_u \propto M_p \tag{4-2}$$

Eq. (4-2) may be rewritten by

$$M_u/M_p = a \tag{4-3}$$

where a is a constant.

However, the actual values of M_u/M_p will not be the same for all available sections since M_u depends on the structural properties of the steel members and concrete slab used. Such changes in the values of M_u/M_p were investigated and simultaneously the value of M_u/M_p is approximated as a function of the plastic moment capacity of given rolled wide flanges, M_p, by using the least square method. The results are as shown in Tables (4-2) through (4-4). The rolled wide flanges used for these calculations are identical to the so-called economy sections which provide a given capacity of plastic moment for the least weight as a steel beam alone.

These sections are tabulated in the handbook of CSA Specification. Table (4-1) lists these sections together with the plastic moment capacities for A36 steel. The thickness of the concrete slab is varied from 3 inches to 6 inches in increments of 1-in. The author assumed the constant yield strength of steel (F_y = 36 Ksi), while various compressive strengths of concrete (f_c' = 3, 3.5 and 4 Ksi) are assumed. The effective width of the concrete slab is assumed as the flange width of the steel beam plus sixteen times the slab thickness. (See Sec. 2.2). The ultimate moment capacity of a composite beam section, M_u, is calculated by Eq. (1-2) and (1-3) which are based solely on the statical equilibrium of internal forces.

These works have been done by using the computer program developed for investigating this problem in a more general manner. It is possible to generate automatically the appropriate value of M_u/M_p as a function of M_p for the arbitrary sections of wide flanges under the conditions of the given thickness of concrete slab, t, yield strength of steel, F_y, and compressive strength of concrete, f_c'. The effective width of the concrete flange is defined according to the assumption in Section 2.2.

From the results shown in Tables (4-2) through (4-4), it is found that the ratio of the ultimate moment capacity of a composite beam to the plastic moment capacity of the corresponding wide flange steel section, M_u/M_p, is likely to lie between 3.0 and 1.0. Also the actual discontinuous relationship between M_u/M_p and M_p has been approximated with the various functions. Among these functions, the following functions present good results for the actual discrete relationship.

$$\text{Type A; } M_u/M_p = \epsilon^{a + b M_p + c M_p^2 + d M_p^3} \quad (M_p \approx 0) \quad (4\text{-}4)$$

$$\text{Type B; } M_u/M_p = a + b (\log M_p) + c (\log M_p)^2 + d (\log M_p)^3 (M_p \neq 0) \quad (4\text{-}5)$$

. . . .

Where the values of coefficients a, b, c, and d corresponding to type A and type B functions are listed in Tables (4-2) through (4-4). Such approximations represent the continuous variation of the ultimate moment capacity of composite beams with plastic moment capacity of steel wide flanges.

Thus Eq. (4-4) and (4-5), compared to Eq. (4-3) will give better results for the approximation of M_u/M_p values, since the latter is expressed as a function of the variable M_p.

However, it is necessary to use the successive linear programming techniques for obtaining minimum weight design solution since the effects of composite action in the beams at the positive moment regions are approximated by the nonlinear functions as in Eqs. (4-4), and (4-5). In the following parts of this section, it is discussed how to apply these nonlinear functions, to the minimum weight design problem which employs the linear programming techniques. For the detailed presentation of the linear optimization procedures, refer to Sec. 2.5.

First it is necessary to assume the initial value of plastic moment capacity, $M_p^{(1)}$, for each composite beam to calculate M_u/M_p ratio, since the optimization problem is confined to the application of linear programming techniques. Such initial values of $M_p^{(1)}$ at composite section which have to be optimized may be estimated from the inspection of the proposed structures, or assumed by the plastic moment capacities of steel beams in which the composite action in the beams is neglected, though the latter ones may be overestimation. It may be started with the average value of M_u/M_p for the available sections as given in Eq. (4-3). Such average values of M_u/M_p calculated for various cases are shown in Tables (4-2) through (4. 4).

In this study, the non-composite structures were also optimized in order to make comparison with the composite structures under the same structural configurations and loading conditions. Hence, for each composite beam section, at which a positive plastic hinge is likely to form, the initial value of M_p/M_u ratio will be approximated by substituting the value of the plastic moment capacity of the corresponding steel beam alone, $M_p^{(1)}$, which obtained as a linear programming solution of the non-composite frame, into either Eq. (4-4) or Eq. (4-5).

Now the elementary mechanisms will be modified at each composite section in which the variable M_u is assumed as equal to the variable M_p multiplied by the M_u/M_p ratio which were calculated previously. Then the new feasible constraints will be generated from these elementary collapse mechanisms where the effects of composite action between steel beams and concrete slab are taken into account.

This yields to a new linear programming solution. Then the convergence of the solution must be checked. This will be done by checking relative errors of the newly obtained ultimate moment capacity of the composite beam, $M_u^{(n)}$, to the previously obtained ultimate moment capacity of the same composite beam, $M_u^{(o)}$ at each composite section considered. It may be written by

$$\text{Relative Error} = (M_{u_i}^{(n)} - M_{u_i}^{(o)}) / M_{u_i}^{(n)} \quad (i = 1, \ldots, \text{NOCP}) \qquad \ldots\ldots\ldots \quad (4\text{-}6)$$

Where NOCP is the number of composite beam sections at which positive plastic hinges are assumed.

If all the relative errors calculated by Eq. (4-6) are satisfactorily small, i.e., within 5%, the correct minimum weight design solution is reached. If not, calculate the new ratio of M_p/M_u for all the composite sections at which positive plastic hinges are assumed, as done previously, and repeat the same procedure until the correct minimum weight design solution is found.

For all the example problems studied in this paper, the cycles of linear programming applications required for obtaining the minimum weight design solution through the foregoing procedures, fall within five times while less than 5% of relative error was secured.

4.3 Effects of Composite Action in a Typical Two-Story Two-Bay Rigid Frame

The frame chosen for the theoretical investigation on the effects of composite action in a rigid frame at ultimate loads, is a two-story two-bay frame with fixed foundations shown in Fig. (4-1).

The object is to compare a composite plastic design to a non-composite plastic design by varying the span length L, loading conditions, the thickness of the concrete slab and the compressive strength of concrete. The results are based on the minimum weight design solutions.

For convenience, it is assumed that the two side-columns are identical. The fully plastic moment capacity of roof-beam, floor-beam, side-columns and center-column are denoted by M_p, M_{p_2}, M_{p_3} and M_{p_4}, respectively. The span length L is varied from 20 ft to 50 ft in increments of 10 ft, and the column height is kept constant at 12 ft. The loading conditions are varied by having the live load to dead load ratio (LL/DL) equal to approximate 1, 2 and 3. The thickness of the concrete slab is varied from 3 inch to 6 inch in increment of 1 inch. The constant yield strength of steel (F_y = 36 Ksi) is assumed in all cases, while various compressive strengths of concrete (f_c' = 3, 3.5 and 4 Ksi) are assumed. The effective width of the concrete slab is assumed as the flange width of the steel beam plus sixteen times the slab thickness.

Based on the slab thickness of 5 inch and a bent spacing of 15 ft, the concrete dead load is about 937 lbs per ft and assuming the beam weight to be around 100 lbs, the total dead load, DL is assumed to be 1 Kip per ft. Based on this load, the total uniform load (w) of 2 K/ft, 3 K/ft and 4 K/ft correspond to the LL/DL ratio of 1, 2 and 3, respectively.

For wind load a conservative value of 50 psf is used, compared to the 20 to 30 psf normally used in multi-story frames. Wind load is replaced by concentrated loads of a 4.5 Kip and 9.0 Kip load at the roof and floor levels, respectively for a column height of 12 ft as shown in Fig. (4-1).

For each plastic design problem, two types of the loading combinations; dead load plus live load and dead load plus live load plus wind load are considered simultaneously through the application of minimum weight design method. According to these loading combinations, working loads are multiplied by the safety factor of 1.7.

In formulating this problem, all the factors used are chosen within practical limits. These works have been carried out systematically, with the aid of a computer.

It is pointed out that in this section mainly the effect of composite action on the beam is investigated since the column remain nearly the same for both the non-composite and the composite design within the range of span length of 20 ft to 30 ft, and in the case of span length of 40 ft or over, the column design is likely to be governed by the axial loads.

One of the most difficult problem in the usual composite design is the determination of the correct beam size. It is desirable to provide composite action in a frame such that it is more useful to be able to obtain a trial section based on the loading conditions and the span length. To do this, an empirical relationship between the plastic moment in a composite beam, the span length and the loading conditions are required. To obtain such a relationship, the example frame of two-story and two-bay building frames are optimized by the foregoing procedures. (See Sections 2.5 and 4.2).

The results are plotted as curves of plastic moments against span lengths for the various loading conditions in Figs. (4-3) and (4-4). In this case, it is assumed that the thickness of the concrete slab is 5 inches and the compressive strength of the concrete 4.0 Ksi. The plastic moments are given in terms of $w L^2$, where w is the working load (Kip/ft) and L the span length (ft) The curves given in Figs (4-3) and (4-4) are the plastic moment capacities of the roof beams, M_{p_1}, and also of the floor beams, M_{p_2}, respectively. For roof beams, the values of composite moment range from $0.0372\ wL^2$ to $0.0432\ wL^2$ for the range of loading of 2 to 4 Kips/ft for L = 20 ft. For L =50 ft the range is $0.0476\ wL^2$ to $0.0521\ wL^2$ for the same range of loading. Values for span lengths between 20 and 50 ft fall within the values given by the above bounds. For floor beams, the changes in the required composite moment capacities are relatively small in comparison with those for roof beams. These curves may be used to estimate a trial steel section for composite design of a rigid frame.

In order to make a comparison, the non-composite design for the corresponding composite case is also carried out. The composite plastic moment capacities of roof and floor beams are expressed as a percentage of the non-composite plastic moment capacities of the corresponding beams, and these are plotted against the span length for the various loading conditions. The relationship obtained are shown in Figs. (4-5) and (4-6). The values of the composite moment of roof beams range from 48.7% to 57.3% of the corresponding non-composite plastic moment for the range of loading of 2 to 4 Kips/ft for L = 20 ft. For L = 50 ft the range is 66.9% to 72.3% for the same range of loading. It can be seen that approximately 30% to 50% reduction for both roof and floor beams in plastic moment capacity or plastic section modulus is possible - larger reduction for the shorter span length of 20 ft and the smaller for the 50 ft span length.

The above investigations have been done for the constant compressive strength of concrete ($f_c' = 4.0$ Ksi). Further the effect due to the change in compressive strength of concrete under the constant loading condition (w= 3.0 Kips/ft) is investigated. The compressive strength of concrete is varied to 3.0, 3.5 and 4.0 Ksi. The results shown in Table (4-5) indicate that when a higher strength of concrete is used, a greater saving in a steel beam is obtained and greater savings for the shorter spans.

It is also studied the effects of the change in the concrete slab thickness upon the required composite plastic moment capacities. The thickness of the concrete slab is varied from 3-in. to 6-in. in increments of 1-in. The results are shown in Figs. (4-7) and (4-8). In each case, the same compressive strength of concrete ($f_c' = 4.0$ Ksi) is assumed and the total live load of 2.0 Kips/ft is assumed. The total dead load for various slab thicknesses are estimated by the following equations.

$$\begin{aligned} DL &= (t/5) \times 0.9 + 0.1 \quad (K/ft) \\ LL &= 2.0 \quad (K/ft) \\ w &= DL + LL \end{aligned} \qquad (4\text{-}7)$$

Where t is the thickness of the concrete slab (inch). Eq. (4-7) is derived to assume that the dead load of the concrete slab of thickness t is proportional to that of a 5-in. concrete slab.

The results show that the greater the thickness of the concrete slab is, the larger the possible reduction in the steel beam is, and also the shorter the span-length, the greater the saving.

From the above it can be seen that there is a decided advantage in using the composite construction from the standpoint of savings in steel.

CHAPTER 5

DESIGN EXAMPLES USING LINEAR PROGRAMMING TECHNIQUES

5.1 Symmetrical Two-Story, Three-Bay Steel Frame with Composite Beams

As a first design example a two-story three-bay frame is considered. The overall dimensions, working loads and member codes are shown in Fig. (5-1). The assumed critical sections are shown in Fig. (5-2). The A36 steel wide flanges and the concrete slab of 5-in. thickness with compressive strength of 4.0 Ksi are to be used.

For design purposes, it is assumed that a 1 Klf uniformly distributed dead load and a 2 Klf uniformly distributed live load act on all the beams, and that wind forces of 4.5 Kip and 9.0 Kip act at the roof and floor levels, respectively. The ultimate loads are assumed to be the working loads multiplied by the safety factor of 1.7.

It is also assumed that the final design will be symmetrical since the wind may act from either direction. In this example frame, four plastic moment capacities of M_{p_1} to M_{p_4} are assigned to the members of the beams and columns as shown in Fig. (5-2). Further the effects of the composite action is taken into account for the elementary beam mechanisms according to the procedures described in Sec. 4.2 in which the location of positive plastic hinges is assumed at mid span for all the beams. Based on the elementary mechanisms of 10 beam mechanisms, 3 sway mechanisms and 12 joint mechanisms, the computer program generated 125 equilibrium equations of feasible mechanisms. In addition, the moment capacity of each column member is constrained by the lower bound as shown in Fig. (5-2). Such a plastic moment capacity as a lower bound is approximately set forth under the following assumption. Each column has the sectional capacity to be able to resist such an axial load as calculated by distributing the ultimate gravity loads proportionally over the corresponding column as for a simple framing building. Thus the column can also accomodate with a certain amount of moment resistant capacity which is assumed as a lower bound.

The result based on the foregoing procedure will be more practical and economical in comparison with those calculated without constraining the column members since smaller capacity for the beams is required for the former case, and even though larger capacity for the columns is obtained, the final refinement on the column members for the axial forces resulting from the critical collapse mechanism usually counteracts such a disadvantage. The linear programming solutions obtained are shown by

$$M_{p_1} = 205.31 \text{ K-ft } (334.69 \text{ K-ft})$$

$$M_{p_2} = 184.42 \text{ K-ft } (286.88 \text{ K-ft})$$

$$M_{p_3} = 92.21 \text{ K-ft } (143.44 \text{ K-ft})$$

$$M_{p_4} = 119.70 \text{ K-ft } (119.70 \text{ K-ft})$$

Where ( ) shows the linear programming solution for the corresponding non-composite steel frame, and these solutions are shown in Fig. (5-3).

From the results shown above, it is seen that smaller steel beam capacities for the composite frame in comparison with the corresponding non-composite frame are required. It is also compared in terms of the ratio of the steel beam moment capacity for the composite frame to that for the corresponding non-composite frame. The ratios for the roof and floor beams are given by 61.3% and 64.3%, respectively.

After the simple inspection of the final tableau of the linear programming solution, the critical mechanism is found to be of the beam mechanisms only as shown in Fig. (5-4). For this critical collapse mechanism with no sway load acting, the moment distributions are checked by the plastic moment distribution method. The results of moment distributions which are shown in Fig. (5-5) prove that the moment nowhere exceeds its moment capacity. Based on these moments and the corresponding shears, member designs are started. Only the exterior columns are adjusted for the axial forces according to the method presented in CSA Specification. Though the interior columns are designed for the axial loads only, the selected column sections have enough moment capacity to resist the possible moments, i. e., moments due to erection or side effects beyond the axial loads acting on. The final design sections are shown below.

16WF40 for roof beams B_1 (21WF55)
16WF36 for floor beams B_2 (18WF50)
12WF40 for exterior columns C_1 (14WF48)
12WF40 for interior columns C_2 (12WF40)

Where ( ) shows the final design section for the corresponding non-composite steel frame, and member layout is shown in Fig. (5-6).

The above sections are found to satisfy the requirements for the local buckling. The deflections at working loads are also found to be within the allowable ranges which are proposed in CSA Specification. For the calculation of these deflections, the displacement method is used. The final design for the composite frame leads to a total weight of 10,680 lb., compared to 13,674 lb. for the non-composite frame. From an economic standpoint, the composite design is more economical than that for the corresponding non-composite design. Based on the weight of the steel beams alone, the savings in steel is 27.7%, and if the saving is based on the total weight of the whole frame, then the saving is reduced to 21.9%.

The computer time required for the linear programming solutions for the composite frame including the corresponding non-composite one is approximately 15 minutes 53.3 seconds on the IBM 360/ Model 65 computer.

5.2 Symmetrical Three-Story Two-Bay Steel Frame with Composite Beams

The second illustrative design is shown in Fig. (5-7). It is a three-story two-bay frame having the overall dimensions, working loads and member codes shown in Fig. (5-7). The assumed critical sections are shown in Fig. (5-8). The A36 steel wide flanges and concrete slab of five inch thickness with compressive strength of 4.0 Ksi are used.

It is assumed that a 1/Klf uniformly distributed dead load and a 2 Klf live load act on all the beams, and that wind forces of 4.5, 9.0 and 9.0 Kip act at the roof, top floor and middle floor levels, respectively. The ultimate loads are assumed to be the working loads multiplied by the safety factor of 1.7. It is also assumed that the final design will be symmetrical since the wind may act from either direction. In this problem, five plastic moment capacities of M_{p_1} to M_{p_5} are assigned to the members of the beams and columns as shown in Fig. (5-8).

Further the effects of the composite action is taken into account for the elementary beam mechanisms according to the procedures described in Sec. 4.2 in which the location of positive plastic hinges is assumed at mid span for all the beams.

Based on the elementary mechanisms of 12 beam mechanisms, 4 sway mechanisms and 14 joint mechanisms, the total 405 equilibrium equations of feasible mechanisms are generated by the computer. As in the first example of Sec. 5.1, the moment capacity of each column is constrained by the lower bound which are shown in Fig. (5-8). Linear programming solutions calculated are presented by

M_{p_1} = 202.64 K-ft (331.16 K-ft)

M_{p_2} = 200.97 K-ft (308.03 K-ft)

M_{p_3} = 208.20 K-ft (308.03 K-ft)

M_{p_4} = 104.10 K-ft (154.01 K-ft)

M_{p_5} = 179.70 K-ft (179.70 K-ft)

Where ( ) shows the linear programming solution for the corresponding non-composite frame,and these solutions are shown in Fig. (5-9).

It is seen that smaller steel beam capacities for the composite frame in comparison with the corresponding non-composite frame are required. The results are also compared in terms of the ratio of the steel beam moment capacity for the composite frame to that for the corresponding non-composite frame. The ratios for the roof, top and middle beams are given by 61.2%, 65.3% and 67.6%, respectively.

The critical mechanism found is composed of the beam and sway mechanisms as shown in Fig. (5-10). The moment distributions for this critical collapse mechanism are checked by the plastic moment distribution method. The results of moment distributions shown in Fig. (5-11) indicate that the moment nowhere exceeds its moment capacity. Based on these moments and the corresponding shears, member designs are started. All the columns are adjusted for the axial forces.

The final design sections are shown below.

16WF40 for roof beams B_1 (21WF55)

16WF40 for top-floor beams B_2 (21WF55)

16WF40 for middle-floor beams B_3 (21WF55)

14WF53 for exterior columns C_1 (16WF58)

16WF88 for interior columns C_2 (16WF88)

Where ( ) shows the sections selected for the corresponding non-composite steel frame, and the member layout is shown in Fig. (5-12). These sections are found to satisfy the requirements for the local buckling. The deflections at working loads are also found to be within the allowable ranges proposed in CSA Specification . The deflections are calculated by the displacement method. The final design for the composite frame leads to a total weight of 14,184 lb., compared to 17,244 lb. for the non-composite frame. Thus based on the weight of the steel beams alone, the savings in steel is 27.3%, and also the saving based on the total weight of the whole frame is 17.9%.

The computer time required for the linear programming solutions for the example composite frames including non-composite one is approximately 52 minutes 53.3 seconds on the IBM 360/Model 65 computer.

CHAPTER 6

REFINEMENTS OF OPTIMUM DESIGN SOLUTION USING NONLINEAR PROGRAMMING TECHNIQUES

6.1 General Remarks

In this chapter, it is investigated the applicability of nonlinear programming techniques to the optimum plastic design of steel frames with composite beams since more general optimization problems can be solved by using nonlinear programming techniques. From among the several available nonlinear programming methods, the gradient projection method is used. (See Sec. 3.2). First the nonlinearity of the constraints and objective function in the optimum design problem will be investigated. Then the general optimization procedures will be shown before entering the example problems.

6.2 Nonlinearity in Plastic Optimum Design Problems

1) Nonlinearity of Plastic Moment Capacity-Unit-Weight Relations

For minimum weight design, the objective function is the total weight of a structure, W, and can be formulated as a function of the weight of each member per foot of length, w_i, as given by Eq. (1-13).

$$W = \sum_{i=1}^{n} w_i \cdot L_i \qquad (1\text{-}13)$$

For solving the minimum weight design problem of steel frames, it is necessary to determine the approximate relationships between the unit weight, w_i, and the plastic moment capacity, M_{p_i}, as written by Eq. (1-14).

$$w_i = f(M_{p_i}) \qquad (1\text{-}14)$$

From the view point of programming problem, the above function can be expressed in any form. For simplicity, however, the unit weight w_i will be taken to be directly proportional to the plastic moment capacity of the member, M_{p_i}, as given by

$$w_i = a\, M_{p_i}^{\,b} \qquad (6\text{-}1)$$

Where a and b are coefficients, and using the least square method, the approximate curve representing the above relationship has been constructed for the so-called economy sections which provides given plastic moment capacity for the least weight. These wide flange sections are listed in Table (4-1) of which plastic moment capacities correspond to A36 steel. The values of coefficients a and b are given by

$$a = 1.3912$$
$$b = 0.6446 \qquad (6\text{-}2)$$

It is noted that Eqs. (6-1) and (6-2) are only the crude approximations of the actual variations of the section properties of selected wide flange members. However, it will be seen that any such approximation is sufficient for the purpose of finding the least weight design from among commercially available sections. The objective function can also take the form of nonlinear cost relationship for the structure. For concrete structures and for steel structures where both rolled and welded sections are used, a cost relationship is more appropriate.

2) Nonlinearity of Equilibrium Equations due to Distributed Loading

In the linear programming solution, it is simply assumed that the plastic hinge forms at mid-span of a beam under the uniformly distributed load. Thus the beam and the sway mechanisms were expressed as the linear function of plastic moment capacities. However where the combined beam and sway mechanism forms, the hinge positions are not predetermined. For the case of a point load on the beam, the sharp change of slope in the bending moment diagram under such a load would restrict possible hinge positions to directly under the load and to the ends of the beam such that the results in the equilibrium equation take linear function of plastic moment capacities. In the case of a distributed load, it is necessary to treat the problem as that of maximizing fully plastic moment. Hence the resulting equilibrium equation includes the partial derivative of unknown plastic moment capacity, or defined by a nonlinear function of plastic moment capacities.

Equilibrium method by Douglas [18] is used to generate the nonlinear constraints for the beam and sway mechanisms under the distributed loads. The application of the equilibrium method is discussed on the example problems in Secs. 6.4 through 6.7.

3) Nonlinearity of Ultimate Moment Capacity of Composite Section-Plastic Moment Capacity of Wide Flange Section Relation

As was noted in Sec. 4.2, the ultimate moment capacity of a composite beam in the positive moment region, M_u, shall be approximated as a function of the plastic moment capacity of a rolled wide flange, M_p, assuming that all other factors related to the ultimate moment capacity of the composite beam are known.

$$M_u = f(M_p) \tag{4.1}$$

Since nonlinear programming techniques are used as an optimum tool, the ultimate moment capacity of a composite beam section, M_u, can be formulated as a nonlinear function of the plastic moment capacity of a rolled wide flange, M_p.

Using the least square method, the value of M_u is approximated as a function of M_p as shown in Tables (6-1) through (6-3). The rolled wide flanges used for these calculations, are identical to so-called economy sections listed in Table (4-1). The thickness of the concrete slab is varied from 3-in. to 6-in. in increments of 1-in. It is assumed the constant yield strength of steel (F_y = 36 Ksi) while various compressive strengths of concrete (f_c' = 3, 3.5 and 4.5 Ksi) are assumed. The effective width of the concrete flange is assumed as the flange width of the steel beam plus sixteen times the slab thickness.

It is noted that the relationship between M_u and M_p can be approximated with the various functions. Among these functions, the following function seems to present sufficient approximations for the actual discrete relationships.

$$M_u = a + b\,M_p + c\,M_p^2 + d\,M_p^3 \qquad (6\text{-}3)$$

Where the value of coefficients a, b, c and d are listed in Tables (6-1) through (6-3). Thus the above relationships are assumed to represent the continuous variation of the ultimate moment capacity of composite beams with plastic moment capacity of steel wide flanges.

6.3 Nonlinear Optimization Procedures

The formulation of nonlinear minimum weight design problem consists of the following steps.

1) Set the problem such that feasible constraints and objective function can be generated.

2) Apply nonlinear programming techniques to obtain the optimum solution.

The nonlinear optimization procedures are carried out by using computer programs which employ nonlinear programming techniques. In order to obtain a computer program which would be applicable to any general type of structure the computation is divided among a main program and several auxiliary subroutines. The function of the main program is to carry out the steps of the gradient projection algorithm as set forth in Sec. 3.2, and also to read in the input data and print out the solution. The functions of the auxiliary subroutines are designed to calculate the values of the constraint functions, to calculate the values of the objective functions and gradient vector, and to calculate the inverse matrix. The main program is applicable to any design problem which has been formulated in terms which correspond to the requirement of the gradient projection method. The auxiliary subroutines may need to be revised for each individual problem, depending on the proposed structure, the constraints and the objective function. A general flow diagram for the main program, and short description of input data are presented in Appendix 4.

6.4 Symmetrical Two-Story Single-Bay Steel Frame

As a first example the structure shown in Fig. (6-1) is considered. It is a two-story single-bay frame having the overall dimensions shown in Fig. (6-1). All the structural members are assumed to be steel alone.

It is assumed that a 2 Klf uniformly distributed load acts on the roof beam and a 3 Klf uniformly distributed load acts on the floor beam, and that wind forces of 4.5 Kip and 10.1 Kip act at the roof and the floor levels, respectively. The ultimate loads are assumed to be the working loads multiplied by the safety factor of 1.7. Since the wind may act from either direction, it is assumed that the final design will be symmetrical. This example involves four plastic moment capacities of M_{p_1} to M_{p_4}.

The idealized structure for optimization is shown in Fig. (6-2). From the 27 feasible collapse mechanisms which are shown in Fig. (6-3), the necessary nonlinear and linear constraints are formulated. In generating such nonlinear constraints the method shown in Sec. 6.2 (2) is employed. For the combined sway and beam mechanisms, the plastic hinges in the roof and floor beams with span length L (30 ft), occur at distance U_1 and U_2, from the right-hand-side origin of the corresponding beam. The following equilibrium equations are generated.

$$a(1) = 4\,M_{p_1} - 0.85\,L^2 \geq 0$$

$$a(2) = 2\,M_{p_1} + 2\,M_{p_3} - 0.85\,L^2 \geq 0$$

$$a(3) = 4\,M_{p_2} - 1.275\,L^2 \geq 0$$

$$a(4) = 2\,M_{p_2} + 2\,M_{p_3} + 2\,M_{p_4} - 1.275\,L^2 \geq 0$$

$$a(5) = 4\,M_{p_3} - 91.8 \geq 0$$

$$a(6) = 2\,M_{p_1} + 2\,M_{p_3} - 91.8 \geq 0$$

$$a(7) = 4\,M_{p_4} - 372.94 \geq 0$$

$$a(8) = 2\,L\,M_{p_2} + 2\,L\,M_{p_3} + 2\,(U_2 + L)\,M_{p_4} - (372.94 + 2.55\,L^2)\,U_2 \geq 0$$

$$\text{where } U_2 = \sqrt{(M_{p_2} + M_{p_3} + M_{p_4})/2.55}$$

$$a(9) = 2\,M_{p_2} + 4\,M_{p_3} + 2\,M_{p_4} - (91.8 + 1.275\,L^2) \geq 0$$

$$a(10) = 2\,L\,M_{p_1} + 2\,(U_1 + L)\,M_{p_3} - (91.8 + 1.7\,L^2)\,U_1 \geq 0$$

$$\text{where } U_1 = \sqrt{(M_{p_1} + M_{p_3})/1.7}$$

$$a(11) = 4LM_{p_1} + 2U_1 M_{p_3} - (91.8 + 1.7L^2)U_1 \geq 0$$

where $U_1 = \sqrt{2M_{p_1}/1.7}$

$$a(12) = 2LM_{p_2} + 2(U_2 + L)M_{p_3} + 2(U_2 + L)M_{p_4} - (464.74 + 2.55L^2)$$

$$U_2 \geq 0$$

where $U_2 = \sqrt{(M_{p_2} + M_{p_3} + M_{p_4})/2.55}$

$$a(13) = 2U_2 M_{p_1} + 2LM_{p_2} + 2LM_{p_3} + 2(U_2 + L)M_{p_4}$$

$$- (464.74 + 2.55L^2)U_2 \geq 0$$

where $U_2 = \sqrt{(M_{p_2} + M_{p_3} + M_{p_4})/2.55}$

$$a(14) = (4L - U_2)M_{p_2} + U_2 M_{p_3} + 3U_2 M_{p_4} - (372.94 + 2.55L^2)$$

$$U_2 \geq 0$$

where $U_2 = \sqrt{2M_{p_2}/2.55}$

$$a(15) = 2LM_{p_1} + 2U_1 M_{p_2} + 2(U_1 + L)M_{p_3} + 2U_1 M_{p_4} - (91.8 + 2.975L^2)$$

$$U_1 \geq 0$$

where $U_1 = \sqrt{(M_{p_1} + M_{p_3})/1.7}$

$$a(16) = 4LM_{p_1} + 2U_1 M_{p_2} + 2U_1 M_{p_3} + 2U_1 M_{p_4} - (91.8 + 2.975L^2)$$

$$U_1 \geq 0$$

where $U_1 = \sqrt{2M_{p_1}/1.7}$

$$a(17) = 3M_{p_2} + 3M_{p_3} + M_{p_4} - (91.8 + 1.275L^2) \geq 0$$

$$a(18) = 2M_{p_1} + 3M_{p_2} + M_{p_3} + M_{p_4} - (91.8 + 1.275L^2) \geq 0$$

$$a(19) = 2\,L\,U_2\,M_{p_1} + 2\,L\,U_1\,M_{p_2} + 2\,L\,(U_1 + U_2)\,M_{p_3}$$

$$+ 2\,(U_1\,U_2 + L\,U_1)\,M_{p_4} - (464.74 + 4.25\,L^2)\,U_1\,U_2 \geq 0$$

where $U_1 = \sqrt{(M_{p_1} + M_{p_3})/1.7}$

$U_2 = \sqrt{(M_{p_2} + M_{p_3} + M_{p_4})/2.55}$

$$a(20) = 4\,L\,U_2\,M_{p_1} + 2\,L\,U_1\,M_{p_2} + 2\,L\,U_1\,M_{p_3} + 2\,(U_1\,U_2 + L\,U_1)\,M_{p_4}$$

$$- (464.74 + 4.25\,L^2)\,U_1\,U_2 \geq 0$$

where $U_1 = \sqrt{2\,M_{p_1}/1.7}$

$U_2 = \sqrt{(M_{p_2} + M_{p_3} + M_{p_4})/2.55}$

$$a(21) = 2\,U_2\,M_{p_1} + 4\,L\,M_{p_2} + 2\,U_2\,M_{p_4} - (464.74 - 2.55\,L^2)\,U_2 \geq 0$$

where $U_2 = \sqrt{2\,M_{p_2}/2.55}$

$$a(22) = 4\,L\,M_{p_2} + 4\,U_2\,M_{p_4} - (372.94 + 2.55\,L^2)\,U_2 \geq 0$$

where $U_2 = \sqrt{2\,M_{p_2}/2.55}$

$$a(23) = 2\,L\,M_{p_1} + 3\,U_1\,M_{p_2} + (U_1 + 2\,L)\,M_{p_3} + U_1\,M_{p_4}$$

$$- (91.8 + 2.975\,L^2)\,U_1 \geq 0$$

where $U_1 = \sqrt{(M_{p_1} + M_{p_2})/1.7}$

$$a(24) = 4\,L\,M_{p_2} + 2\,U_2\,M_{p_3} + 2\,U_2\,M_{p_4} - (464.74 + 2.55\,L^2)\,U_2 \geq 0$$

where $U_2 = \sqrt{2\,M_{p_2}/2.55}$

$$a(25) = 4\,L\,M_{p_1} + 3\,U_1\,M_{p_2} + U_1\,M_{p_3} + U_1\,M_{p_4} - (91.8 + 2.915\,L^2)$$

$$U_1 \geq 0$$

$$\text{where } U_1 = \sqrt{2\,M_{p_1}/1.7}$$

$$a(26) = 2\,L\,U_2\,M_{p_1} + 4\,L\,U_1\,M_{p_2} + 2\,L\,U_2\,M_{p_3} + 2\,U_1\,U_2\,M_{p_4}$$

$$- (464.74 + 4.25\,L^2)\,U_1\,U_2 \geq 0$$

$$\text{where } U_1 = \sqrt{(M_{p_1} + M_{p_3})/1.7}$$

$$U_2 = \sqrt{2\,M_{p_2}/2.55}$$

$$a(27) = 4\,L\,U_2\,M_{p_1} + 4\,L\,U_1\,M_{p_2} + 2\,U_1\,U_2\,M_{p_4} - (464.74 + 4.25\,L^2)$$

$$U_1\,U_2 \geq 0$$

$$\text{where } U_1 = \sqrt{2\,M_{p_1}/1.7}$$

$$U_2 = \sqrt{2\,M_{p_2}/2.55}$$

According to the procedure presented in Sec. 6.2 (1) the nonlinear objective function is defined by $F = -W$, in which W is the total weight.

Starting the algorithm at the point $\overline{M}_{p_o}$ (400.0, 600.0, 400.0, 300.0), the gradient projection procedure leads to the minimum weight solution $\overline{M}_{p_{min}}$ (214.89, 325.87, 214.89, 110.98) for this problem. The active constraints at the point $\overline{M}_{p_{min}}$ are a(8), a(14), a(19), a (20), a(26) and a(27). The solutions are as follows:

M_{p_1} = 214.89 K-ft (191.25)

M_{p_2} = 325.87 K-ft (332.46)

M_{p_3} = 214.89 K-ft (191.25)

$M_{p_4} = 110.98$ K-ft (141.21)

Where ( ) shows the corresponding linear programming solution in which all the constraints and the objective function are approximated as the linear function of variables, M_{p_i}.

The computer time required for this solution was 14.3 seconds on IBM System 360/Model 65 computer.

6.5 Unsymmetrical One-Story Two-Bay Steel Frame

The second example considered is an unsymmetrical one-story two-bay steel frame having the overall dimensions shown in Fig. (6-4). The solutions are sought for the steel frame alone in which the effects of the composite action are neglected.

It is assumed that a 2 Klf uniformly distributed load acts on the beam, and that wind force of 5.63 Kip acts at the roof level. The ultimate loads are assumed as the working loads multiplied by the safety factor of 1.7. In this example the wind forces from both directions are considered since the structure is not symmetrical. The plastic moment capacity of each member is numbered as shown in Fig. (6-5). Thus this example involves three plastic moment capacities of M_{p_1} to M_{p_3}. Corresponding to the 12 feasible collapse mechanisms which are shown in Fig. (6-6), the nonlinear and linear constraints are formulated. For the combined sway and beam mechanisms, assuming the plastic hinges in the left-hand-side beam with span length L_1 (35 ft) and the right-hand-side beam with span length L_2 (25 ft), to occur at distance U_1 and U_2, from the right-hand-side origin of the corresponding beam respectively, the following equilibrium equations are generated.

$$a(1) = 4\,M_{p_1} - 0.85\,L_1^2 \geq 0$$

$$a(2) = 3\,M_{p_1} + M_{p_3} - 0.85\,L_1^2 \geq 0$$

$$a(3) = 4\,M_{p_2} - 0.85\,L_2^2 \geq 0$$

$$a(4) = 3\,M_{p_2} + M_{p_3} - 0.85\,L_2^2 \geq 0$$

$$a(5) = 6\,M_{p_3} - 143.442 \geq 0$$

$$a(6) = (4\,L_1 - U_1)\,M_{p_1} + 5\,U_1\,M_{p_3} - (143.44 + 1.7\,L_1^2)\,U_1 \geq 0$$

where $U_1 = \sqrt{2\,M_{p_1}/1.7}$

$$a(7) = (4\,L_1 - U_1)\,M_{p_1} + U_1\,M_{p_2} + 4\,U_1\,M_{p_3} - (143.44 + 1.7\,L_1^2)\,U_1 \geq 0$$

where $U_1 = \sqrt{2\,M_{p_1}/1.7}$

$$a(8) = (4\,L_2 - U_2)M_{p_2} + 5\,U_2\,M_{p_3} - (143.44 + 1.7\,L_2^2)\,U_2 \geq 0$$

where $U_2 = \sqrt{2\,M_{p_2}/1.7}$

$$a(9) = 2\,M_{p_1} + M_{p_2} + 2\,M_{p_3} - 0.85\,L_1^2 \geq 0$$

$$a(10) = 2\,L_1\,M_{p_1}\,(2\,L_1 - U_1)\,M_{p_2} + (4\,U_1 + 2\,L_1)\,M_{p_3} - (143.44 + 1.7\,L_1^2)$$

$$U_1 \geq 0$$

where $U_1 = \sqrt{(M_{p_1} + M_{p_2} + M_{p_3})/1.7}$

$$a(11) = 2\,L_1\,M_{p_1} + 2\,L_1\,M_{p_2} + (3\,U_1 + 2\,L_1)M_{p_3} - (143.44 + 1.7\,L_1^2)$$

$$U_1 \geq 0$$

where $U_1 = \sqrt{(M_{p_1} + M_{p_2} + M_{p_3})/1.7}$

$$a(12) = 2\,L_1\,U_2\,M_{p_1} + 4\,L_2\,U_1\,M_{p_2} + (3\,U_1\,U_2 + 2\,L_1\,U_2)\,M_{p_3}$$

$$-[143.44 + 1.7\,(L_1^2 + L_2^2)]\,U_1\,U_2 \geq 0$$

$$\text{where } U_1 = \sqrt{(M_{p_1} + M_{p_3})/1.7}$$

$$U_2 = \sqrt{2\,M_{p_2}/1.7}$$

The nonlinear objective function, which is defined as the total weight of the structure, is used as in the first example problem.

Beginning the gradient projection algorithm at the point $\overline{M}_{p_0}$ = (800.0, 500.0, 200.0), the solution $\overline{M}_{p_{min}}$ (428.22, 189.44, 25.28) is obtained. The active constraints at the point $\overline{M}_{p_{min}}$ are a(8), a(10) and a(12). Since three constraints are active, and since the problem is three-dimensional, the point $\overline{M}_{p_{min}}$ corresponds to a unique point which is a vertex of the feasible region. The solution is shown by

M_{p_1} = 428.22 K-ft (390.25)

M_{p_2} = 189.44 K-ft (189.04)

M_{p_3} = 25.28 K-ft (35.86)

Where ( ) shows the corresponding linear programming solution.

The computer time required for this solution was approximately 11.3 seconds on IBM System 360/ Model 65 computer.

6.6 Symmetrical Two-Story Two-Bay Steel Frame with Composite Beams

The third example is a two-story two-bay composite steel frame having the overall dimensions shown in Fig. (6-7). In this example, the effects of composite action are considered according to the procedure shown in Sec. 6.2 (3).

It is assumed that a 2 Klf and a 3 Kfl uniformly distributed loads act on the roof and the floor beams, respectively, and that wind forces of 4.5 Kip and 10.1 Kip act at the roof and the floor levels, respectively. The ultimate loads are assumed to be the working loads multiplied by the safety factor of 1.7. Since the wind may act from either direction, it is assumed that the final design will be symmetrical. The design variables of M_{p_1} to M_{p_4} are assumed. The idealized structure for optimization is shown in Fig. (6-8). From the 32 likely collapse mechanisms, the necessary nonlinear and linear constraints are formulated.

Beginning the gradient projection algorithm at the point $\overline{M}_{p_o}$ = (300, 500, 200, 200), the solution $\overline{M}_{p_{min}}$ = (142.13, 259.42, 15.30, 62.16) is obtained. The active constraints at the point $\overline{M}_{p_{min}}$ are a(3), a(8), a(11) and a(17). These constraints are expressed by

$$a(3) = 2\,L\,U_3\,M_{u_1} + (2\,L\,U_3 - U_1\,U_3)\,M_{p_1} + 2\,L\,U_1\,M_{u_2}$$

$$+ (2\,L\,U_1 - U_1\,U_3)\,M_{p_2} + 4\,U_1\,U_3\,M_{p_3} + 5\,U_1\,U_3\,M_{p_4}$$

$$- (464.74 + 4.25\,L^2)\,U_1\,U_3 \geq 0$$

$$\text{where } U_1 = \sqrt{(M_{u_1} + M_{p_1})/1.7}$$

$$U_3 = \sqrt{(M_{u_2} + M_{p_2})/2.55}$$

$$a(8) = 6\,M_{p_3} - 91.8 \geq 0$$

$$a(11) = 6\,M_{p_4} - 372.94 \geq 0$$

$$a(17) = 2\,L\,(U_1\,U_3 + U_2\,U_3)\,M_{u_1} + 2\,L\,U_2\,U_3\,M_{p_1} + 2\,L\,U_1\,U_2\,M_{u_2}$$

$$+ (2\,L\,U_1\,U_2 - U_1\,U_2\,U_3)\,M_{p_2} + 2\,(U_1\,U_2\,U_3 + L\,U_1\,U_3)\,M_{p_3}$$

$$+ 5\,U_1\,U_2\,U_3\,M_{p_4} - (464.74 + 5.95\,L^2)\,U_1\,U_2\,U_3 \geq 0$$

where $U_1 = \sqrt{(M_{u_1} + M_{p_1})/1.7}$

$U_2 = \sqrt{(M_{u_1} + M_{p_3})/1.7}$

$U_3 = \sqrt{(M_{u_2} + M_{p_2})/2.55}$

and $L = 30$ ft

The ultimate moment capacity of composite section, M_{u_i}, is given by Eq. (6-3), and the critical mechanisms corresponding to the active constraints are shown in Fig. (6-9). The solution is shown as follows.

M_{p_1} = 142.13 K-ft (130.69)

M_{p_2} = 259.42 K-ft (226.28)

M_{p_3} = 15.30 K-ft (45.90)

M_{p_4} = 62.16 K-ft (65.41)

Where ( ) shows the corresponding linear programming solution. From this example it is noted that the nonlinear programming technique of the gradient method is specially powerful when it is applied to the optimum plastic design of composite frames since the problem is confined to the mathematical programming technique, and it will be well compared to the linear programming method shown in Chapters 4 and 5 in which the convergence checks beyond the mathematical solution are required.

The computer time required for this solution was approximately 18.2 seconds on the IBM System 360/ Model 65 computer.

6.7 Unsymmetrical Steel Frame with Composite Beams

The fourth and last example is an unsymmetrical composite steel frame which is shown in Fig. (6-10). In this example, the effects of composite action are considered as in Sec. 6-6.

The working loads are assumed as shown in Fig. (6-10). The ultimate loads are assumed to be the working loads multiplied by the safety factor of 1.7. The design variables of M_{p_1} to M_{p_4} are assumed. The idealized structure for optimization is shown in Fig. (6-11). From the 26 likely collapse mechanisms, the necessary nonlinear and linear constraints are formulated.

Beginning the gradient projection algorithm at the point $\bar{M}_{p_0}$ = (300, 500, 300, 200), the solution $\bar{M}_{p_{min}}$ = (114.59, 243.97, 114.59, 62.16) is obtained. The active constraints at the point $\hat{M}_{p_{min}}$ are a(1), a(2), a(3) and a(4). These constraints are expressed by

$$a(1) = 2\,M_{u_1} + 2\,M_{p_1} - 0.85\,L_1^{\,2} \geq 0$$

$$a(2) = 2\,M_{u_1} + 2\,M_{p_3} - 0.85\,L_1^{\,2} \geq 0$$

$$a(3) = 6\,M_{p_4} - 372.94 \geq 0$$

$$a(4) = 2\,L_1\,U_2\,M_{u_1} + 2\,L_1\,U_1\,M_{u_2} + (2\,L_1\,U_1 - U_1\,U_2)\,M_{p_2}$$

$$+ (U_1\,U_2 + 2\,L_1\,U_2)\,M_{p_3} + 5\,U_1\,U_2\,M_{p_4} - (464.74 + 4.25\,L_1^{\,2})$$

$$U_1\,U_2 \geq 0$$

where $U_1 = \sqrt{(M_{u_1} + M_{p_3})/1.7}$

$U_2 = \sqrt{(M_{u_2} + M_{p_2})/2.55}$

and $L_1 = 30$ ft

$L_2 = 25$ ft

The ultimate moment capacity of composite section, M_{u_1}, is given by Eq. (6-3), and the critical mechanisms corresponding to the above constraints are shown in Fig. (6-12). The solution is shown by

M_{p_1} = 114.59 K-ft (115.65)

M_{p_2} = 243.97 K ft (235.08)

M_{p_3} = 114.59 K-ft (115.65)

M_{p_4} = 62.16 K-ft (62.16)

Where ( ) shows the corresponding linear programming solution. In this example, relatively small modification in the solution is performed by employing the nonlinear programming technique since both the linear and nonlinear solutions are controled by the similar pattern of failure mechanisms. However the refinement by using the nonlinear programming techniques in the minimum weight plastic design appears to be great in such cases as found in the previous examples in which the nonlinear solutions are controled by the different failure mechanisms from those in the linear programming ones. And also it can be said that the critical failure mechanism found by the application of linear programming technique is not always the same for the nonlinear solution, thus more critical failure mechanism and solution can be realized by employing the nonlinear programming techniques.

The computer time required for this solution was approximately 12.7 seconds on the IBM System 360/ Model 65 computer.

CHAPTER 7
CONCLUSION

As a result of the application of the programming techniques to the minimum weight design of composite steel frames, the following conclusions can be presented.

1. Problems of the minimum weight plastic design of a steel frame with composite beams can be formulated as both linear and nonlinear programming problems. The simplex method is used from among the available linear programming methods for the linear optimization, because of its simplicity and flexibility. On the other hand, it is noted that the gradient projection method provides an efficient numerical technique for solving nonlinear programming problems in connection with minimum weight plastic design problems.

2. In order to formulate the minimum weight plastic design of the composite steel frames as a programming problems, it is necessary to obtain the approximate relationship between the ultimate moment capacity of a composite beam and the plastic moment capacity of the corresponding steel beam. It is possible to do this for the available wide flange sections.

3. The minimum weight plastic design of the steel frames with composite beams can be carried out by the successive applications of the conventional minimum weight plastic design method for rigid frames. The developed computer program can generate, automatically, the linear constraints of the feasible collapse modes for the proposed structure and the linear objective function, then produces the linear optimum

solution.

4. The application of the nonlinear programming techniques to the minimum weight plastic design of rigid steel frames, including the composite steel frames, made it possible to eliminate some of difficulties and arbitrary restrictions which are found in the previous minimum weight plastic design method which employs linear programming techniques.

From the results of the plastic design of the composite steel frames, the following conclusion can be drawn.

1. Comparisons of the changes in required plastic moment capacities and relative cost on a weight saving basis of a composite steel frame to that of the corresponding non-composite steel frame, made with several model frames, indicate that there is a definite advantage in using composite construction from either economy or structural overload capacity.

2. Two design examples of the composite frames show that the saving in steel, based on the total weight of the structure, is approximately 20%. Thus, from the economic point of view, less steel is required for the same loads and spans when composite construction is adopted.

3. In addition, these exaple designs show that smaller beam-depths for the composite frame, in comparison with the corresponding non-composite frame, are required. It is noted that a further advantage of the composite construction, of the reduction in overall floor-depths, is possible. This aspect is of particular importance in tall buildings.

4. From the investigation carried out on the effects of changing the thickness of the concrete slab, in the typical two-story two-bay composite frames, upon the required plastic moment capacities of the steel beams, it can be seen that the greater the thickness of the concrete slab is, the larger the possible reduction in the steel beams, and also the shorter the span-lengths, the greater the savings in steel.

5. Similarly, the investigation on the effects of changing the compressive strength of the compressive strength of the concrete, under the constant loading condition, shows that the higher the strength of the concrete is, the smaller the required steel beam is, and also the shorter the span-lengths, the greater the savings in steel.

APPENDIX I

FIGURES

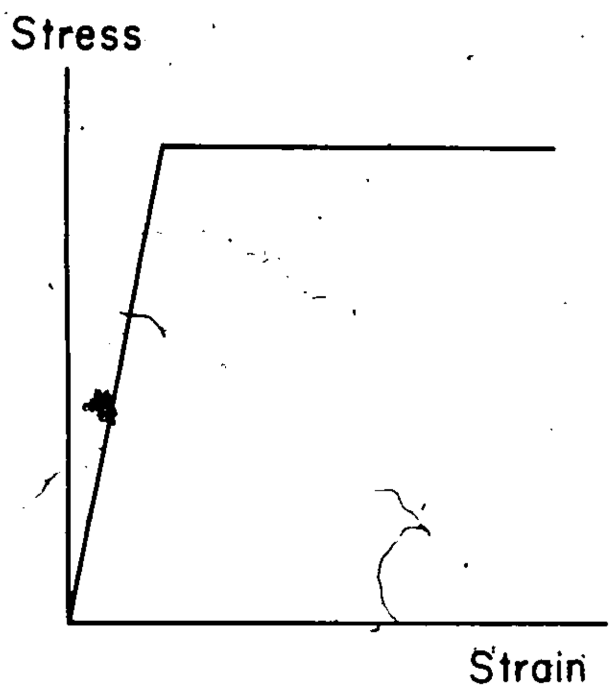


Fig. I-1 Idealized Stress-Strain Curve

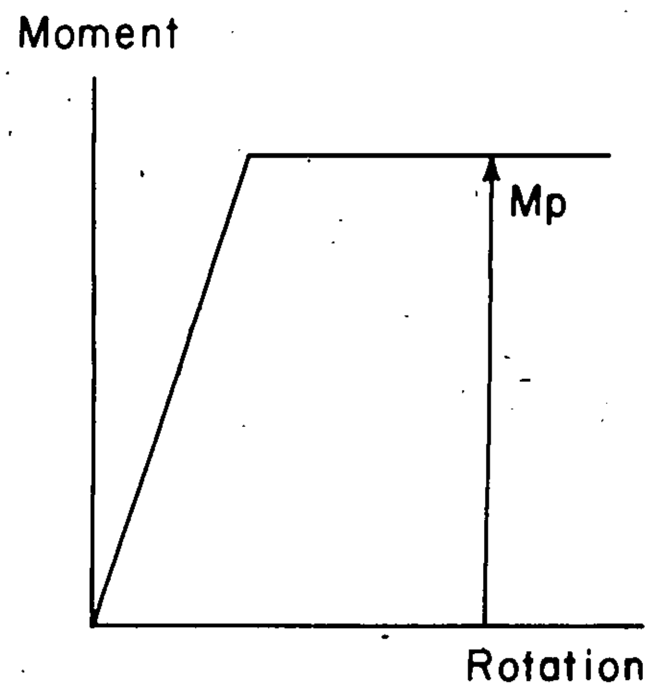


Fig. I-2 Moment-Rotation Relationship

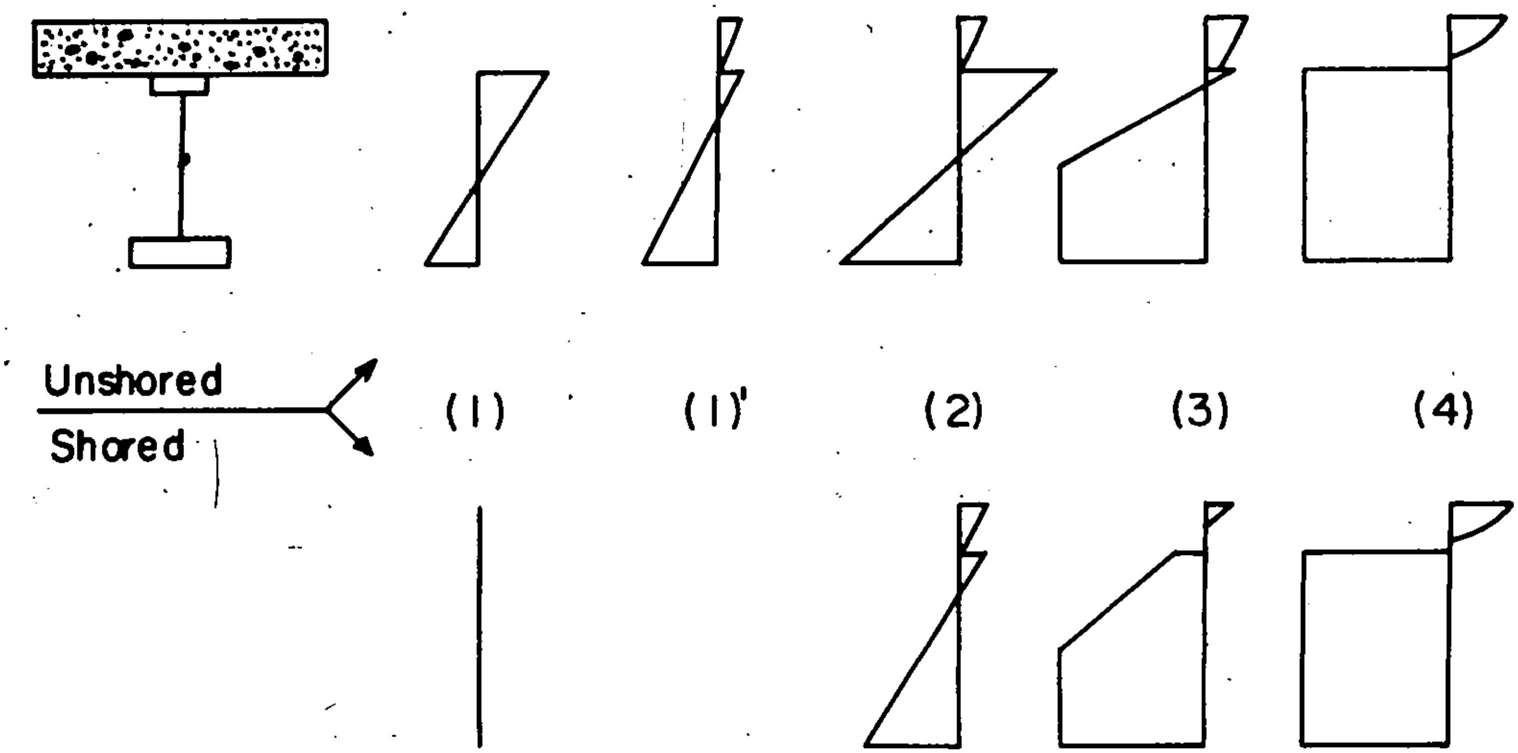


Fig. 1-3 Shored Composite Girders and Unshored Ones

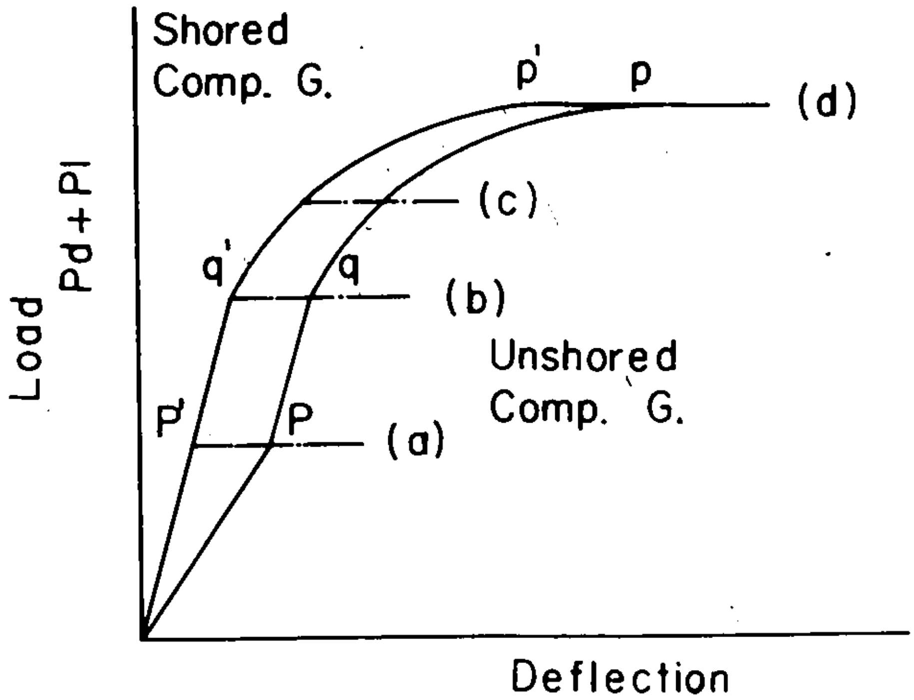


Fig. 1-4 Load-Deflection Curves for Composite Girders

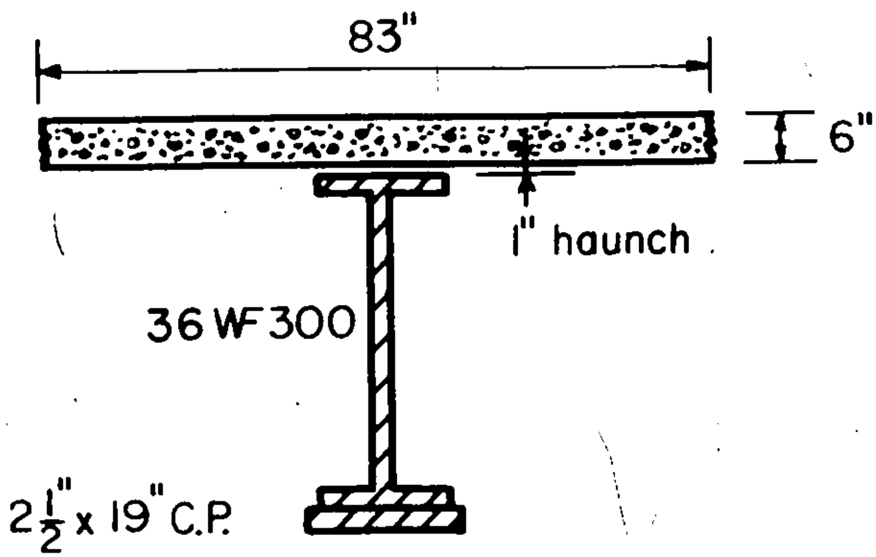


Fig. 1-5 Specimens

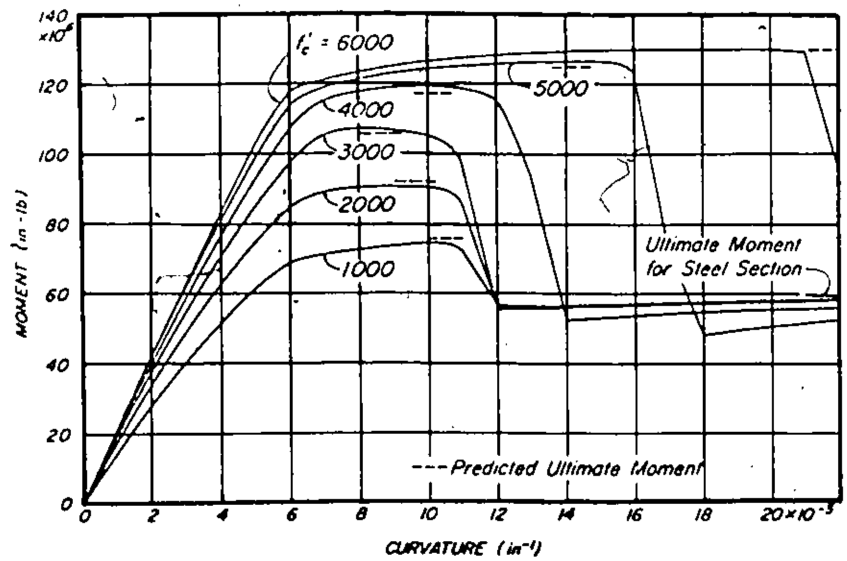


Fig. I-6 Moment-Curvature Curve for Different Strengths of Concrete

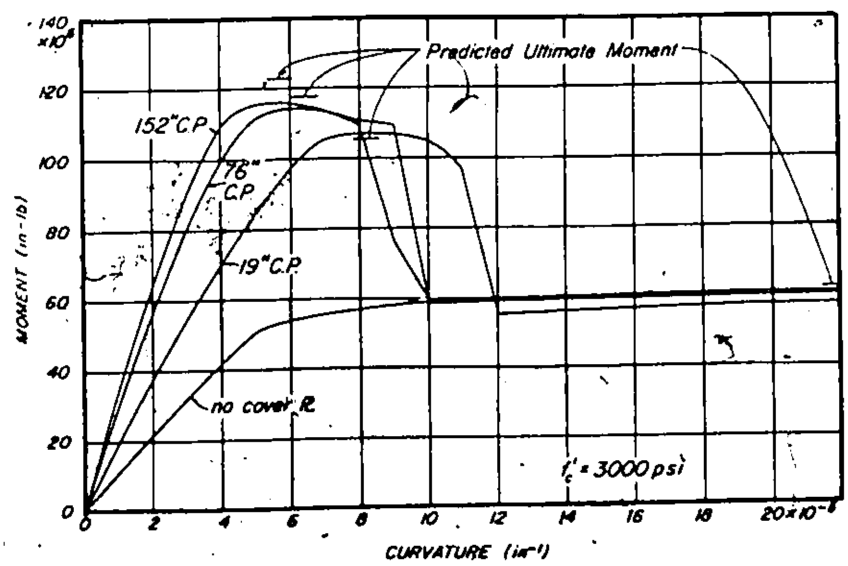


Fig. I-7 Moment-Curvature Curve for Different Widths of Cover Plates

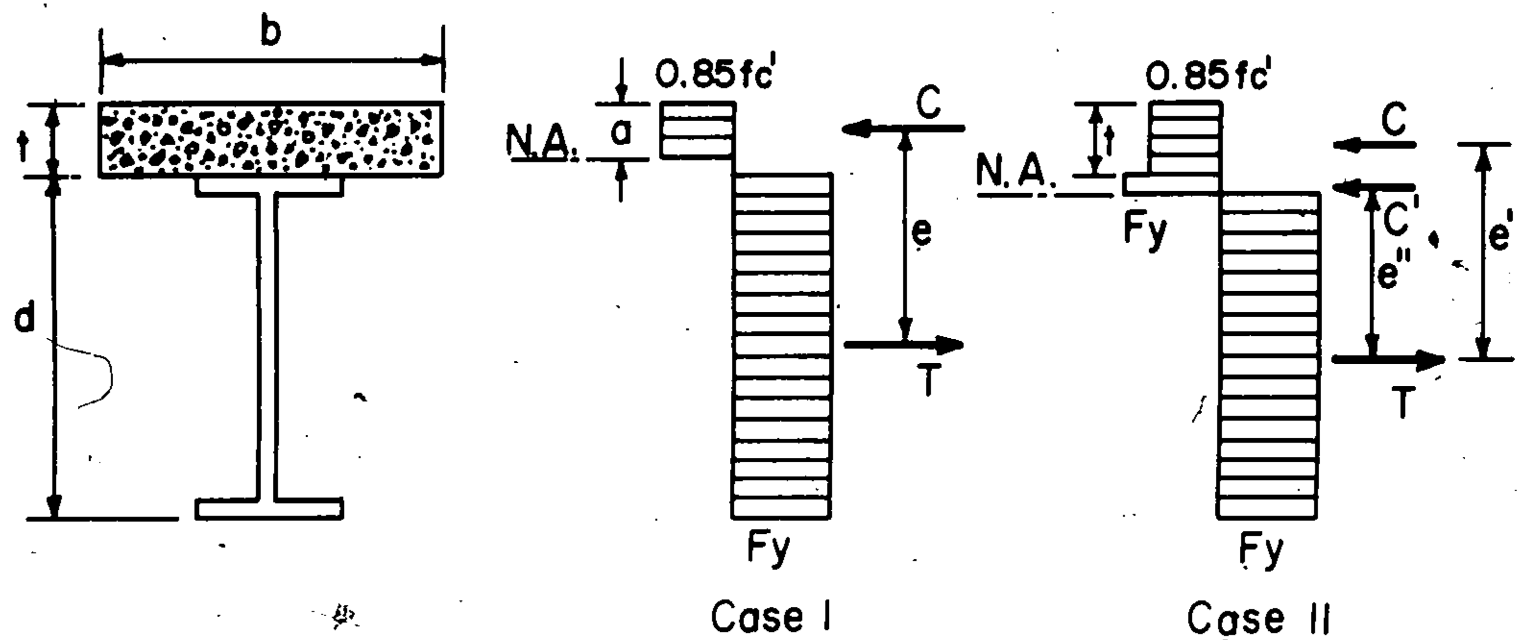


Fig. I-8 Stress Distribution at Ultimate Moment

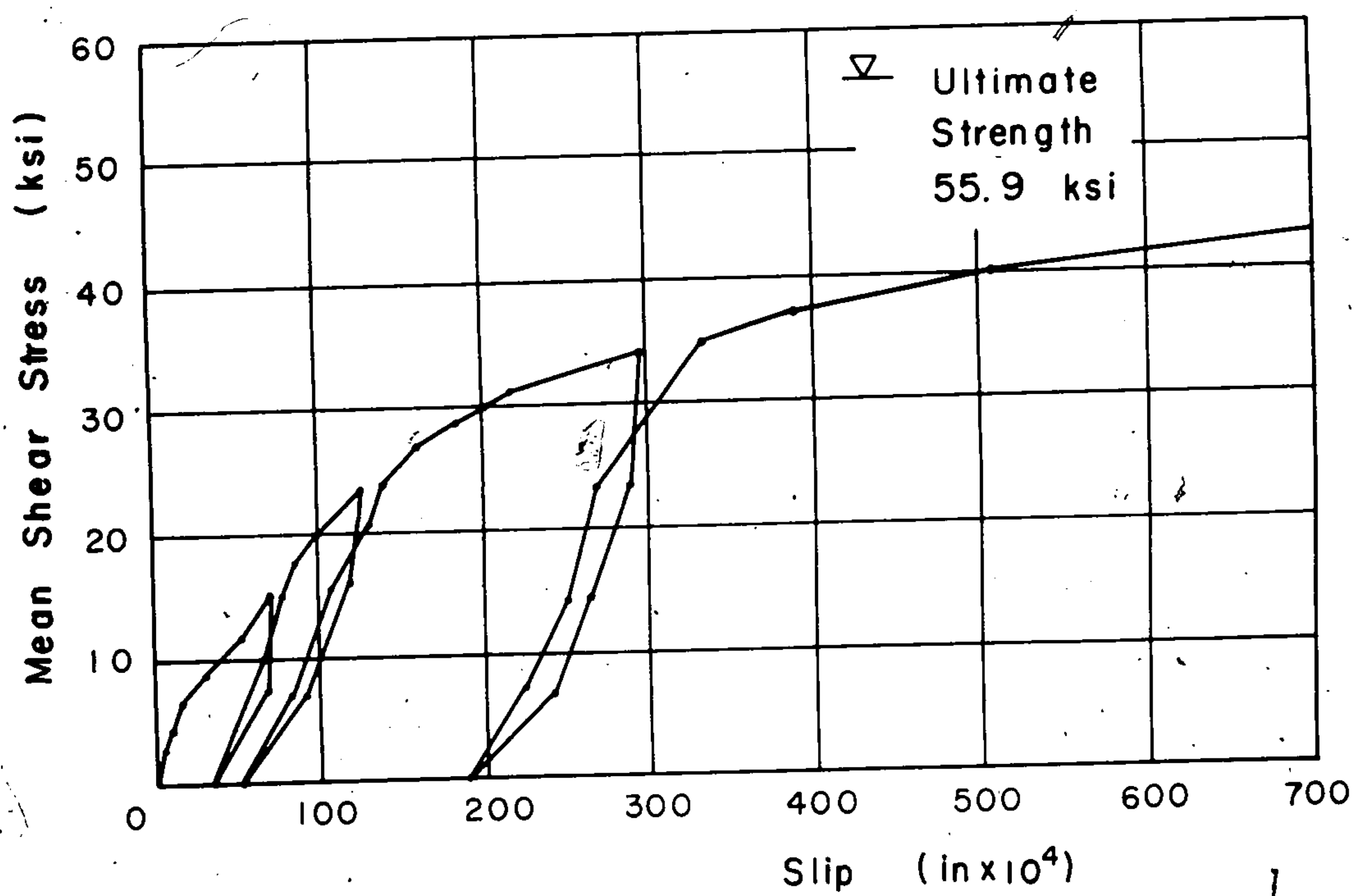


Fig. 1-9 Load versus Slip Curve

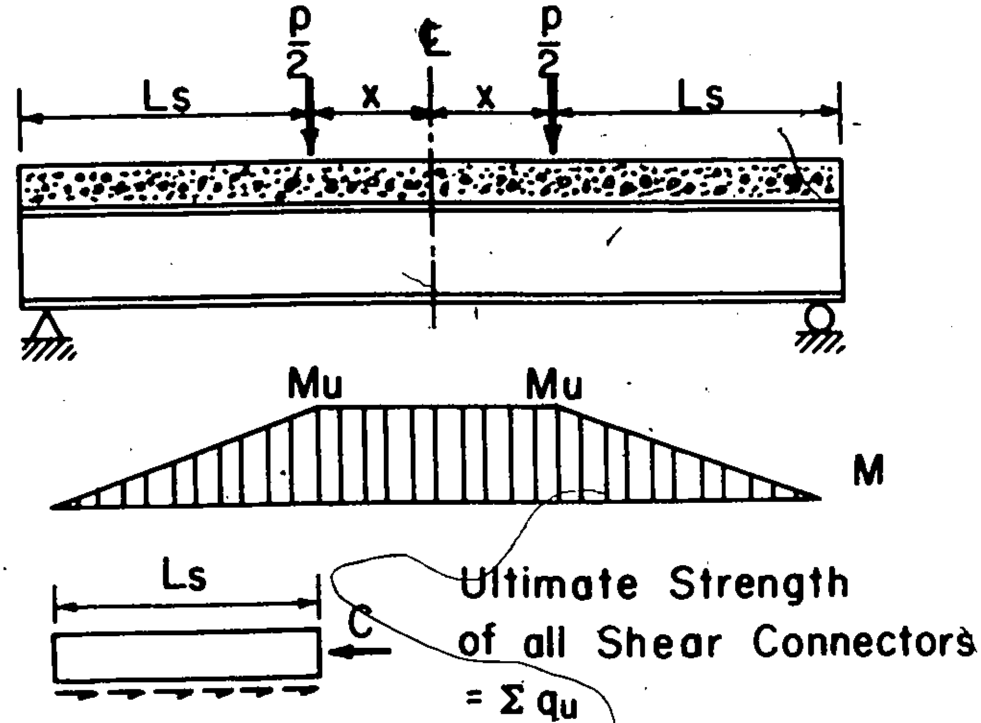


Fig. I-10 Shear Connector Forces at Ultimate Moment

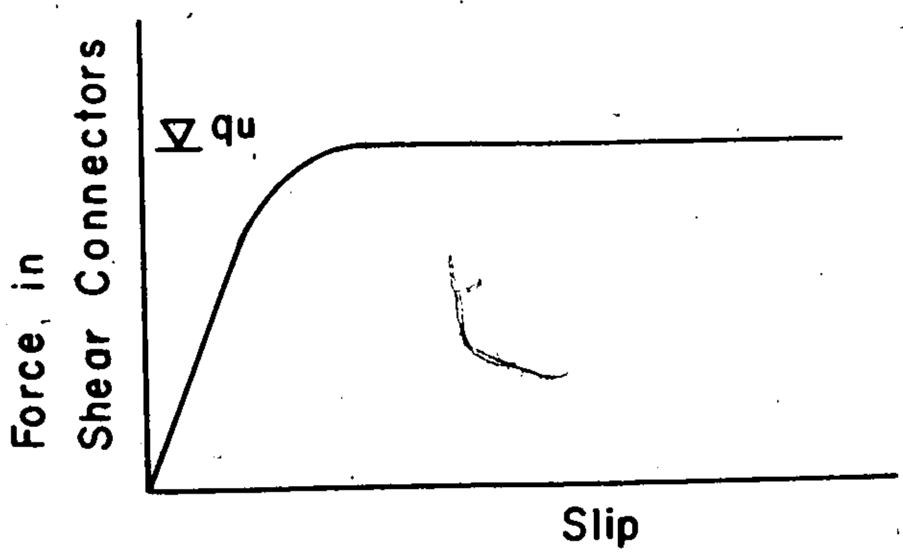


Fig. I-11 Idealized Load-Slip Curve

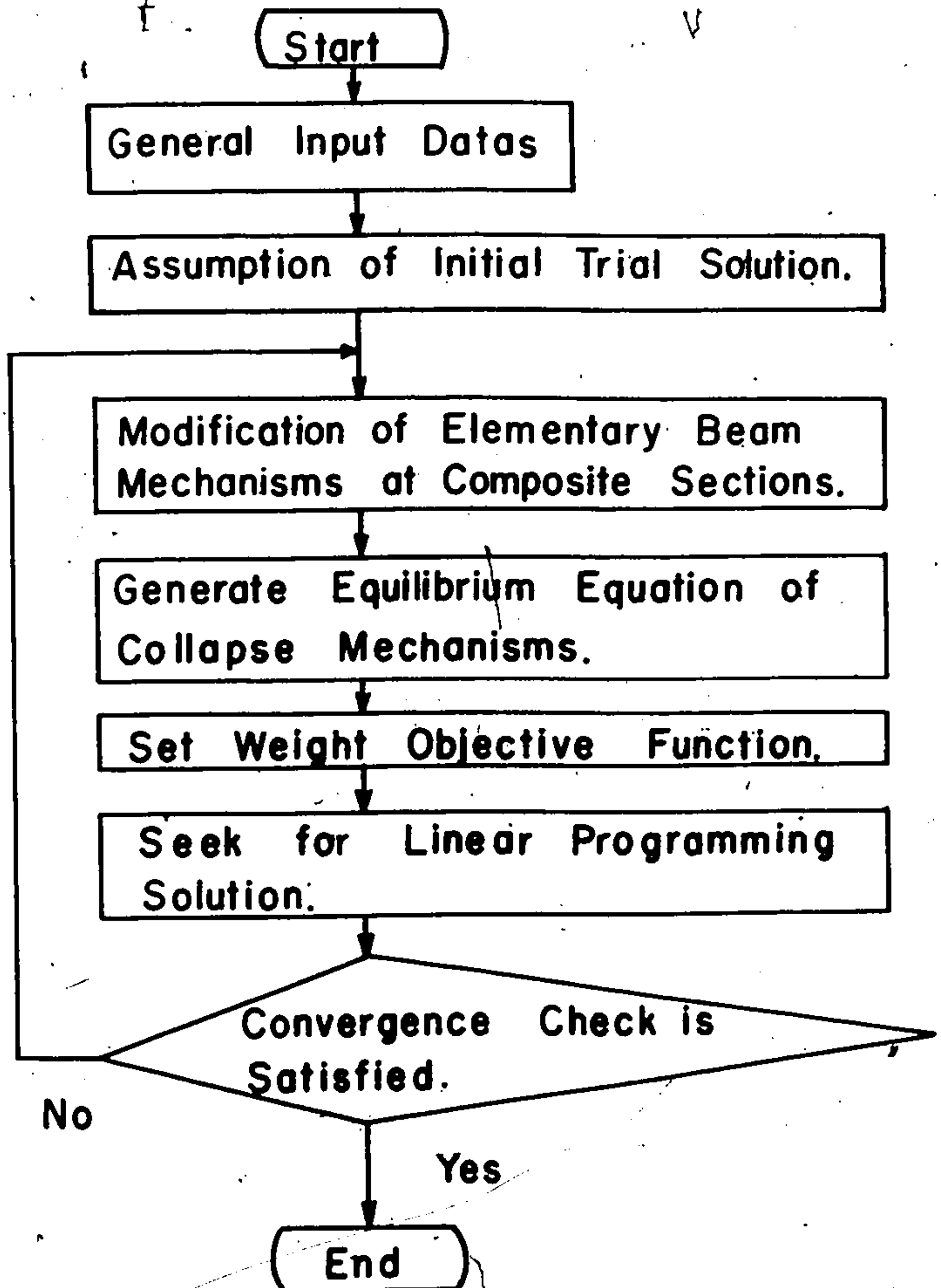


Fig. 2-1 Operations in Optimization of Composite Steel Frames

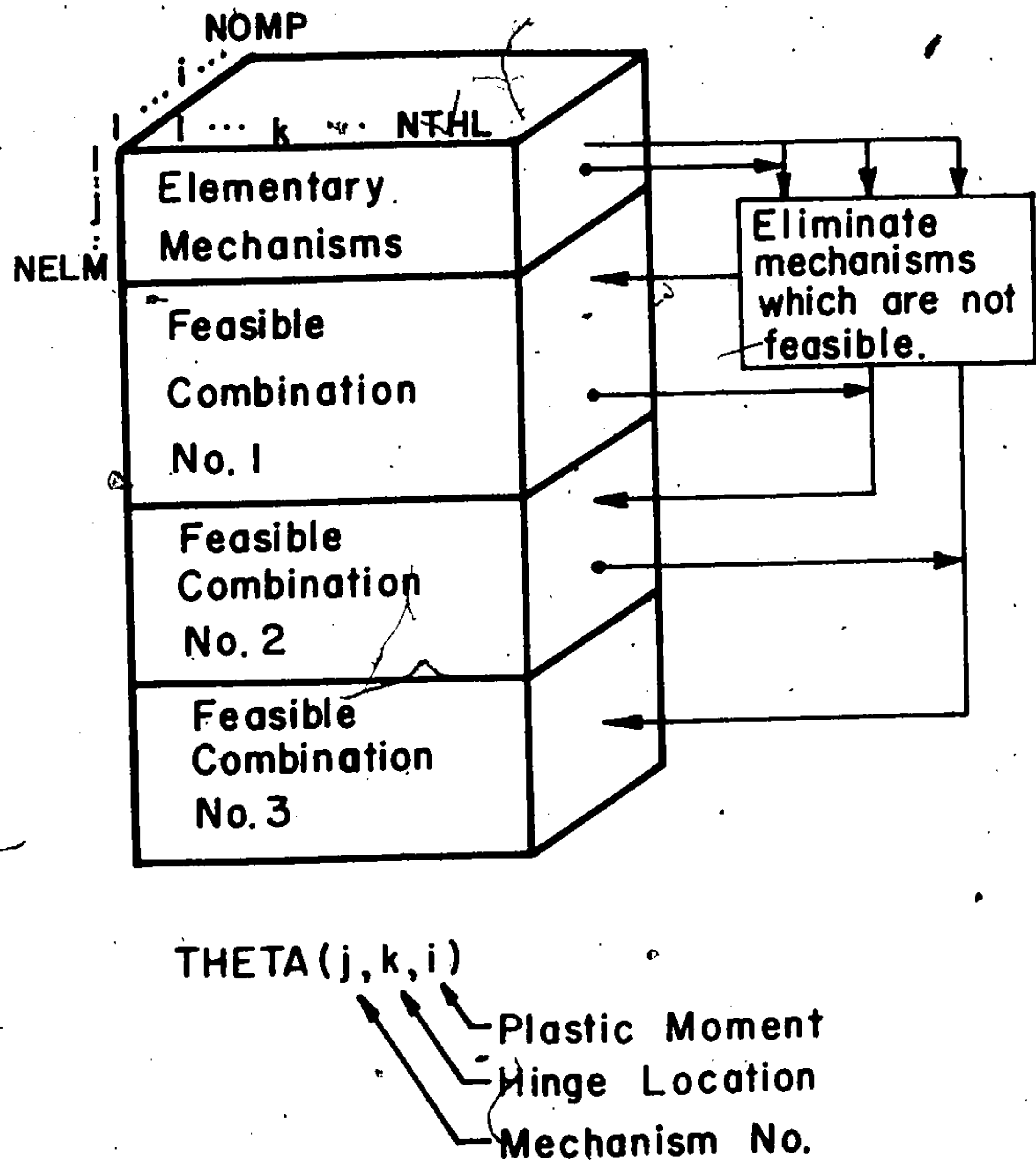
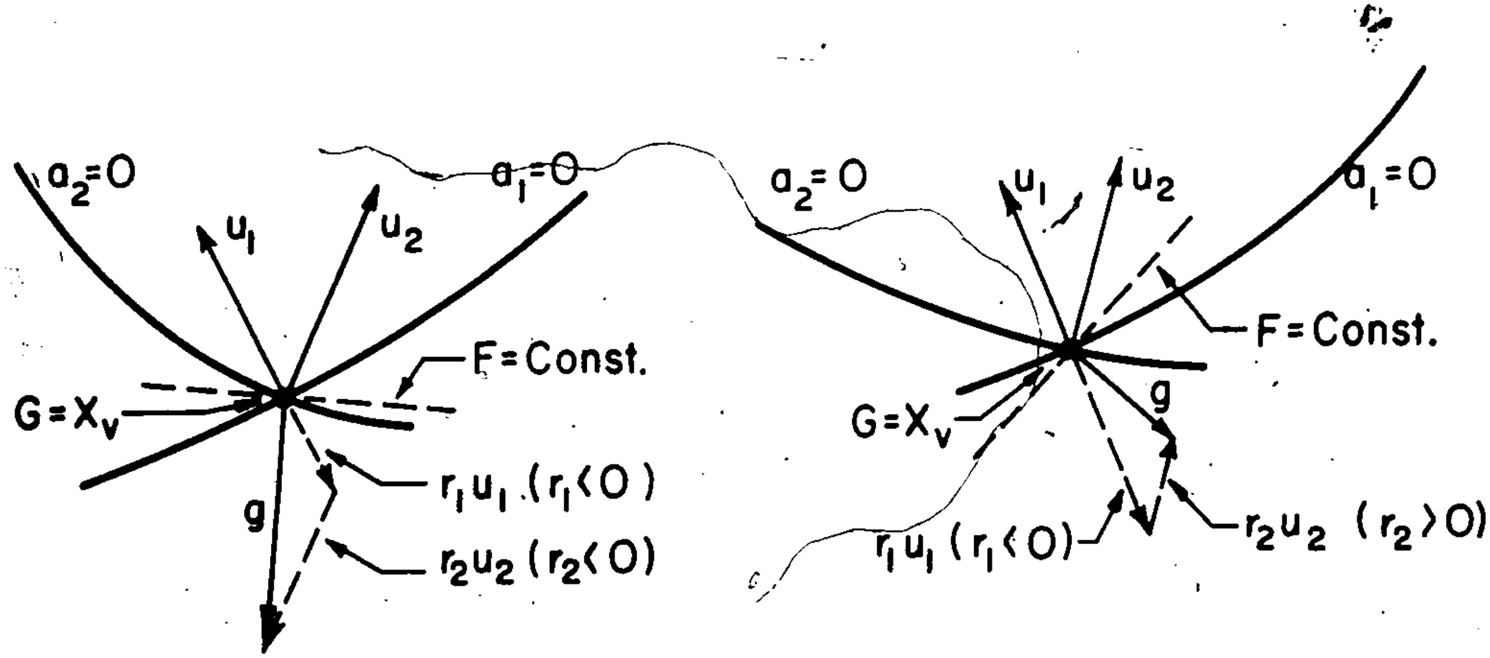


Fig. 2-2 Generation of Feasible Mechanisms



(a) $X_v = X_{max}$ (b) $X_v \neq X_{max}$

Fig. 3-1 Conditions for Local Optimum

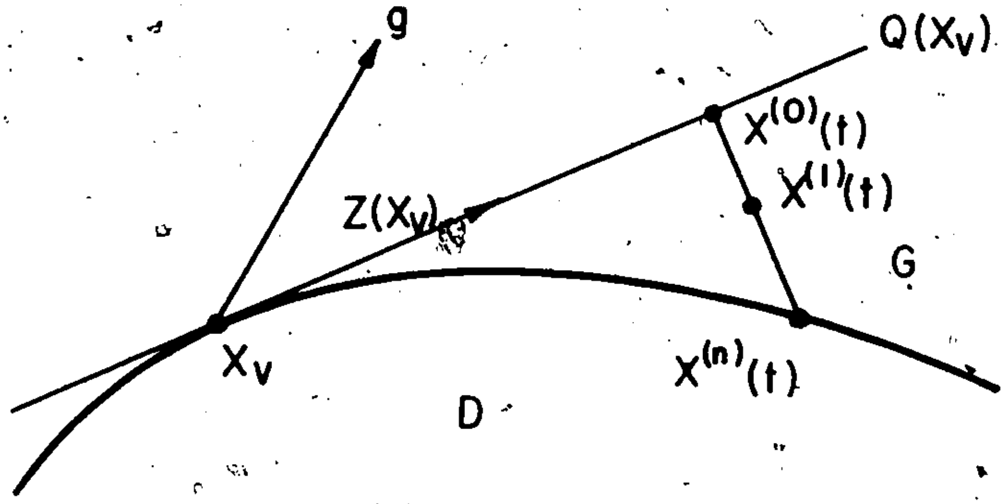


Fig. 3-2 Iteration to Obtain Feasible Point X_{v+1} in Intersection G

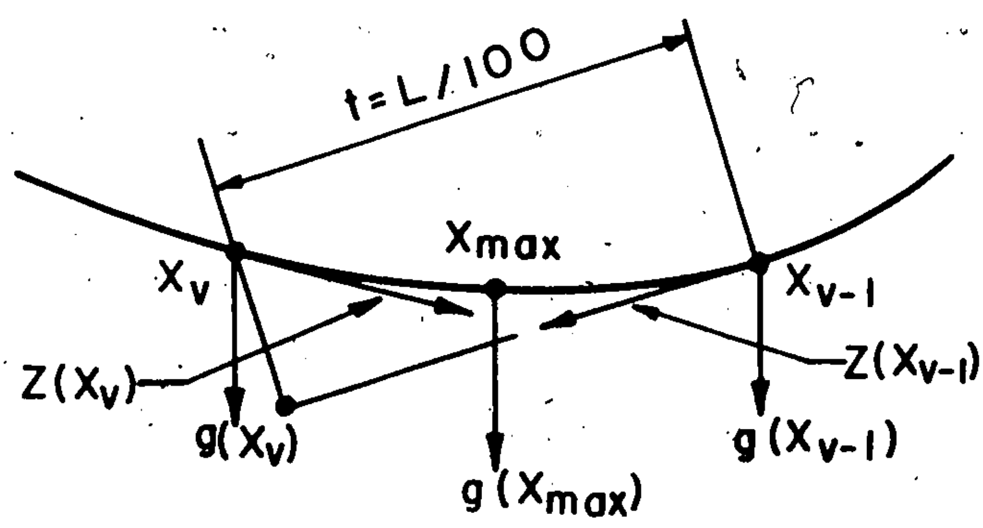


Fig. 3-3 Solution Point not at Vertex of Feasible Region

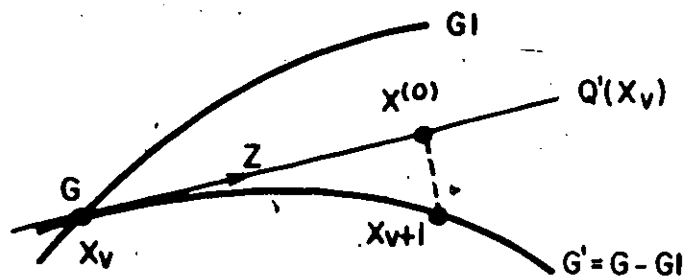


Fig. 3-4 Determination of Point X_{v+1} in Intersection G' but not in G1

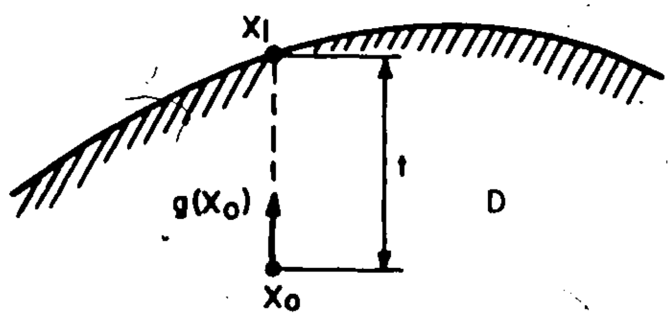


Fig. 3-5 Interpolation to Obtain Point X_1 on B

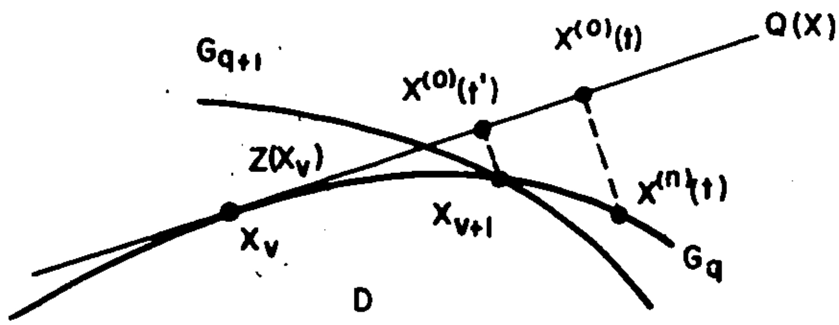


Fig. 3-6 Interpolation to Obtain Feasible Point X_{v+1}

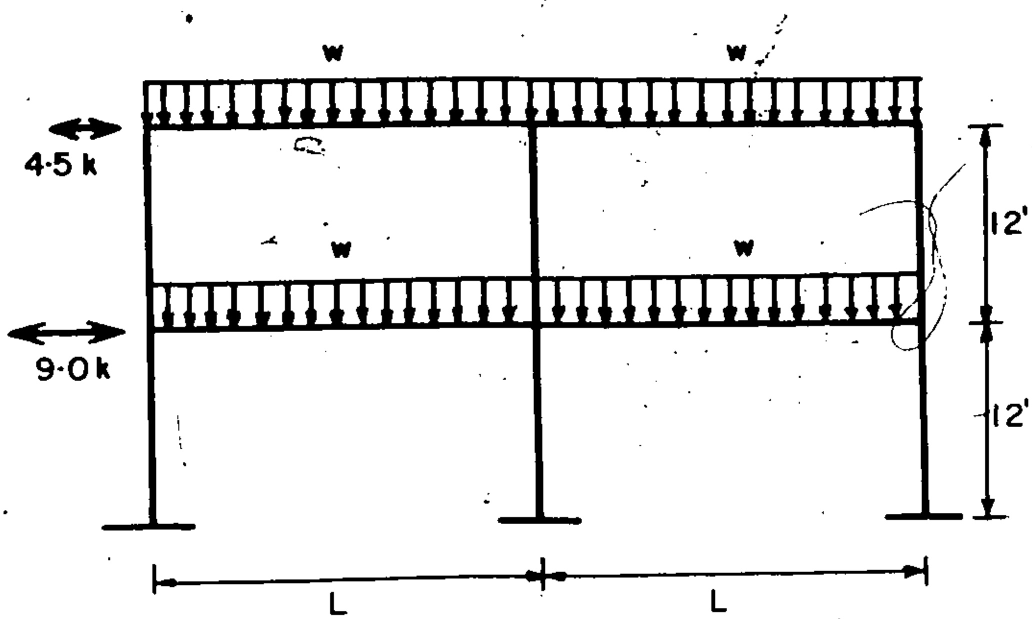


Fig. 4-1 Symmetrical Two-Story Two-Bay Frame Showing Working Loads

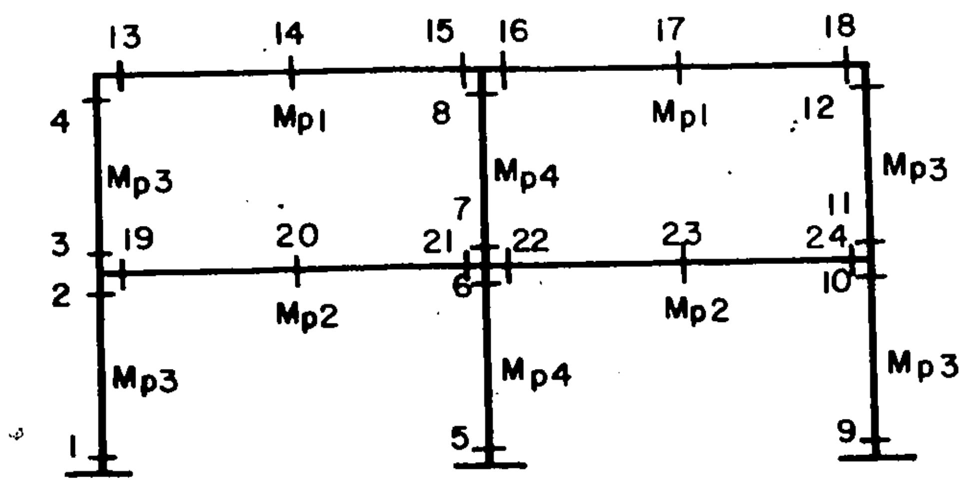


Fig. 4-2 Idealized Symmetrical Two-Story and Two-Bay Frame

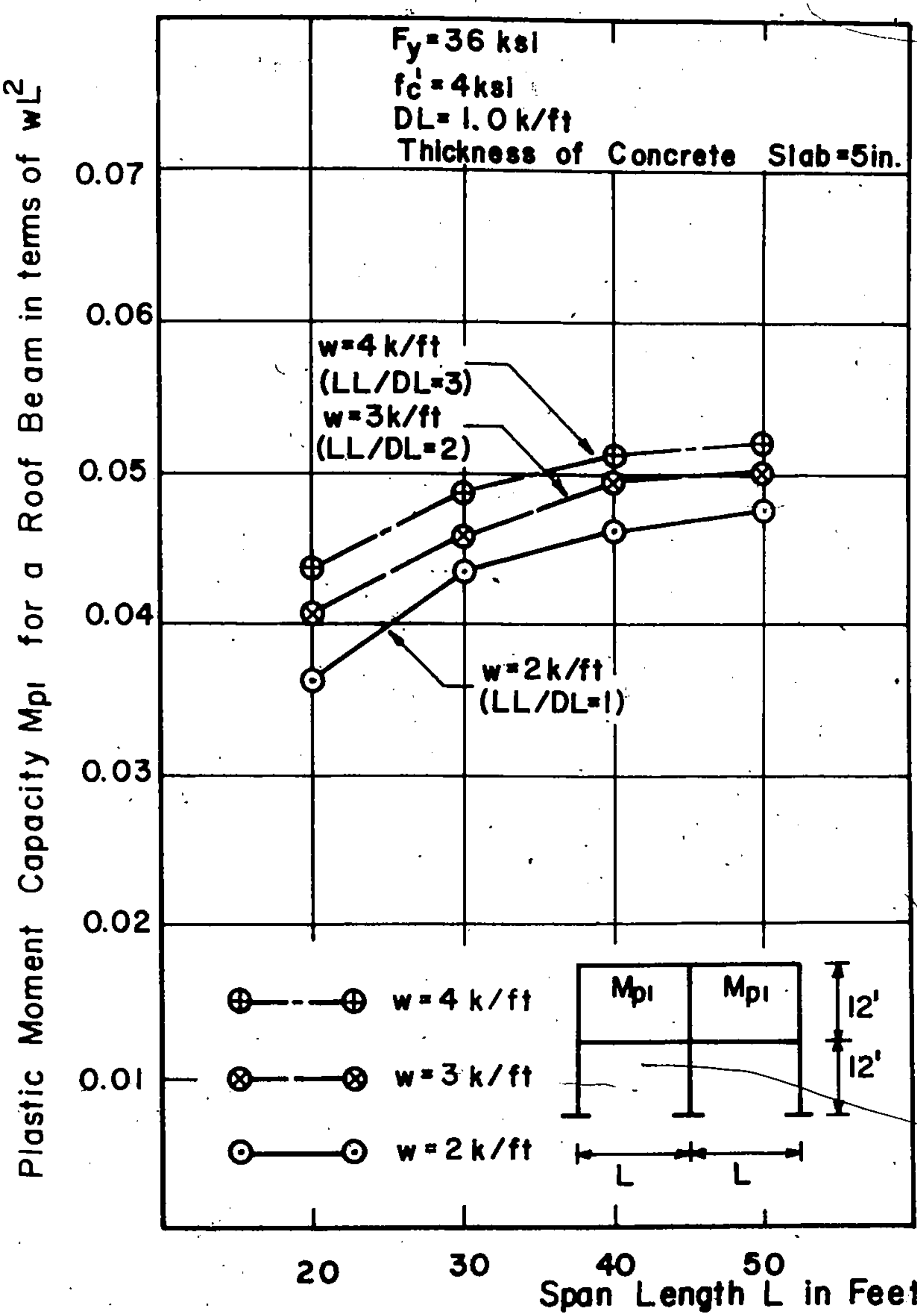


Fig. 4-3 Empirical Relationship between Plastic Moment Capacity, Loading and Span Length for a Roof Beam of Two-Story Two-Bay Example Frame

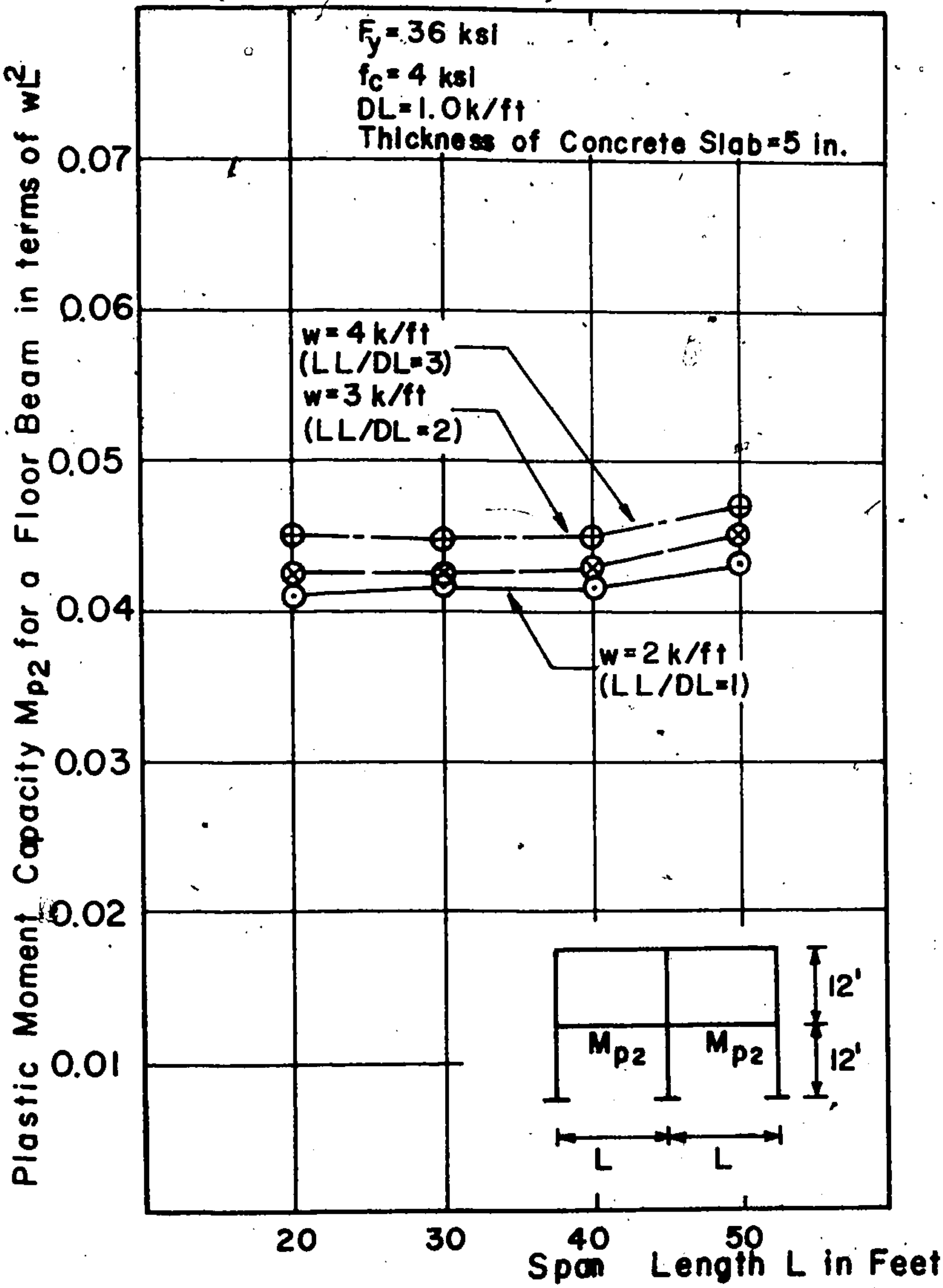


Fig. 4-4 Empirical Relationship between Plastic Moment Capacity, Loading and Span Length for a Floor Beam of Two-Story Two-Bay Example Frame

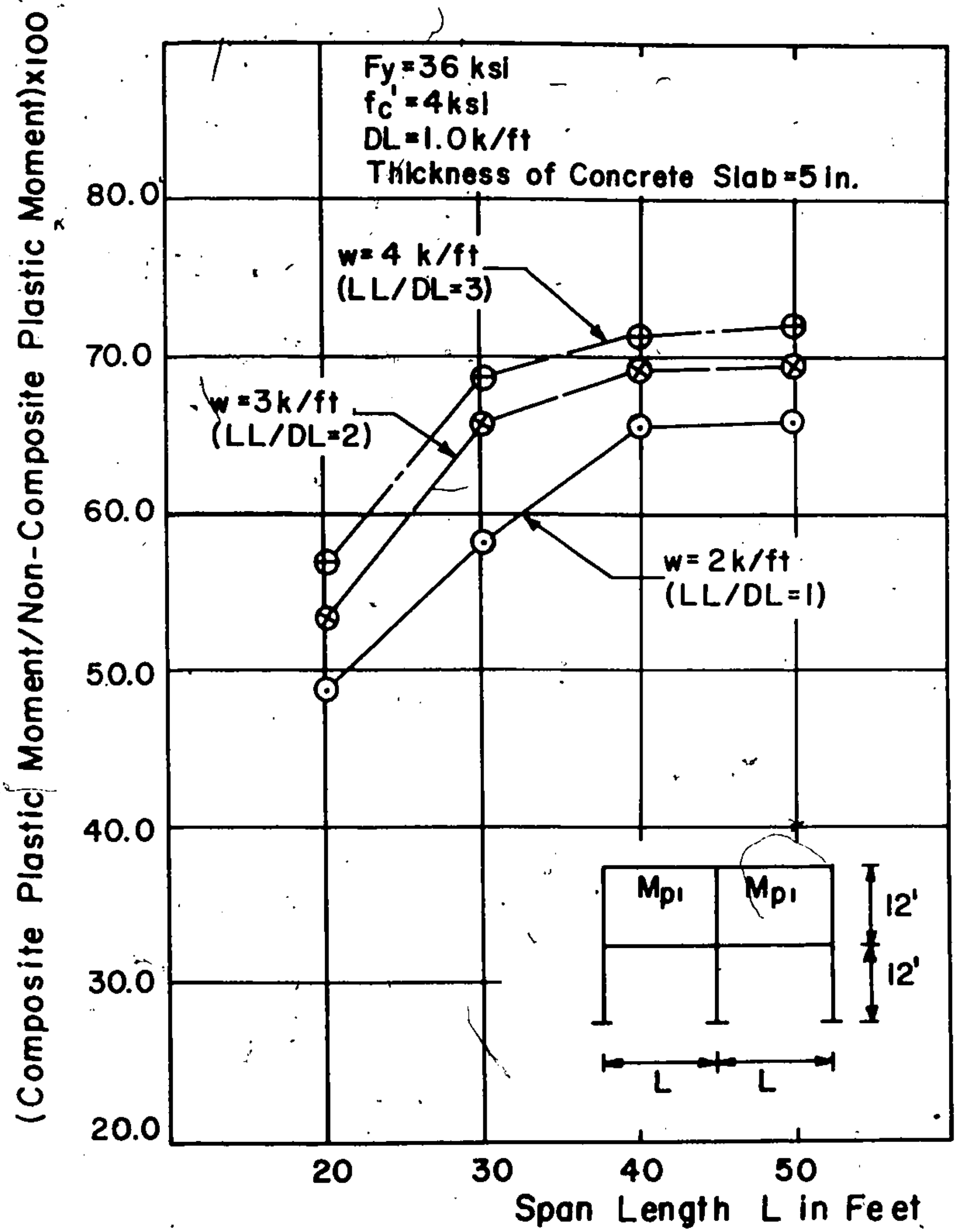


Fig.4-5 Empirical Relationship between Composite Plastic Moment/Non-Composite Plastic Moment, Loading and Span Length for a Roof Beam of Two-Story Two-Bay Example Frame

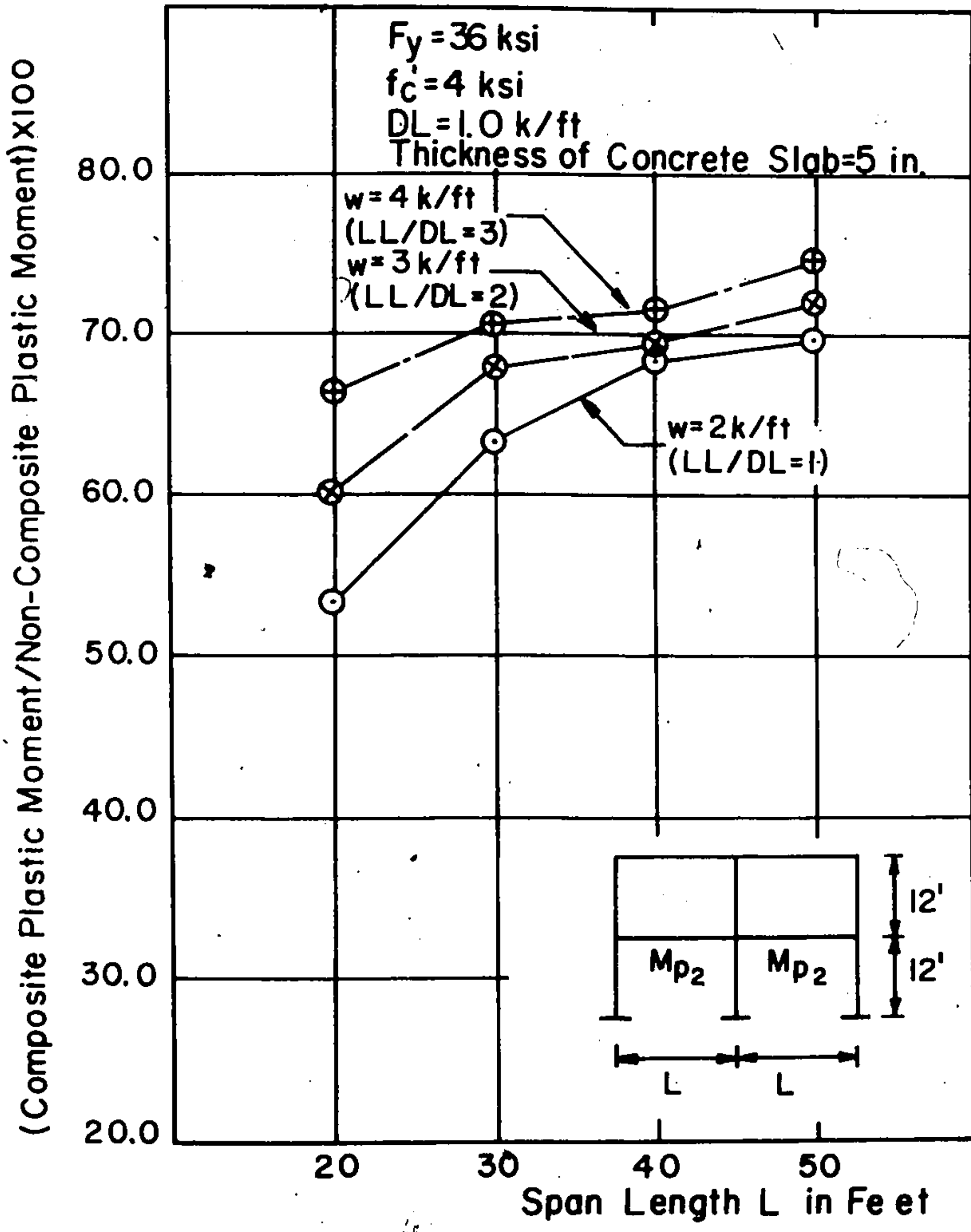


Fig. 4-6 Empirical Relationship between Composite Plastic Moment/Non-Composite Plastic Moment, Loading and Span Length for a Floor Beam of Two-Story Two-Bay Example Frame

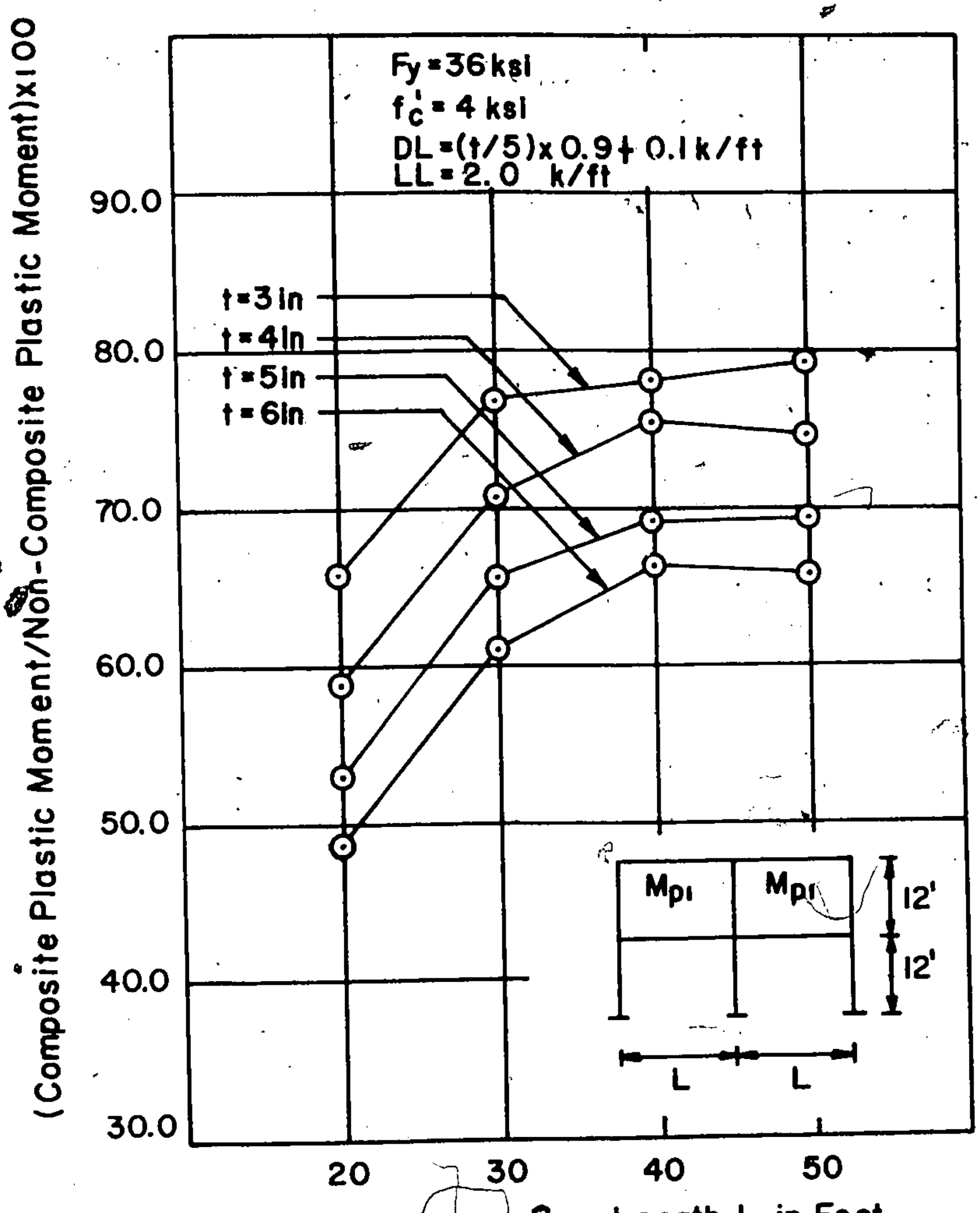


Fig. 4-7 Empirical Relationship between Composite Plastic Moment/Non-Composite Plastic Moment, Slab Thickness and Span Length for a Roof Beam of Two-Story Two-Bay Example Frame

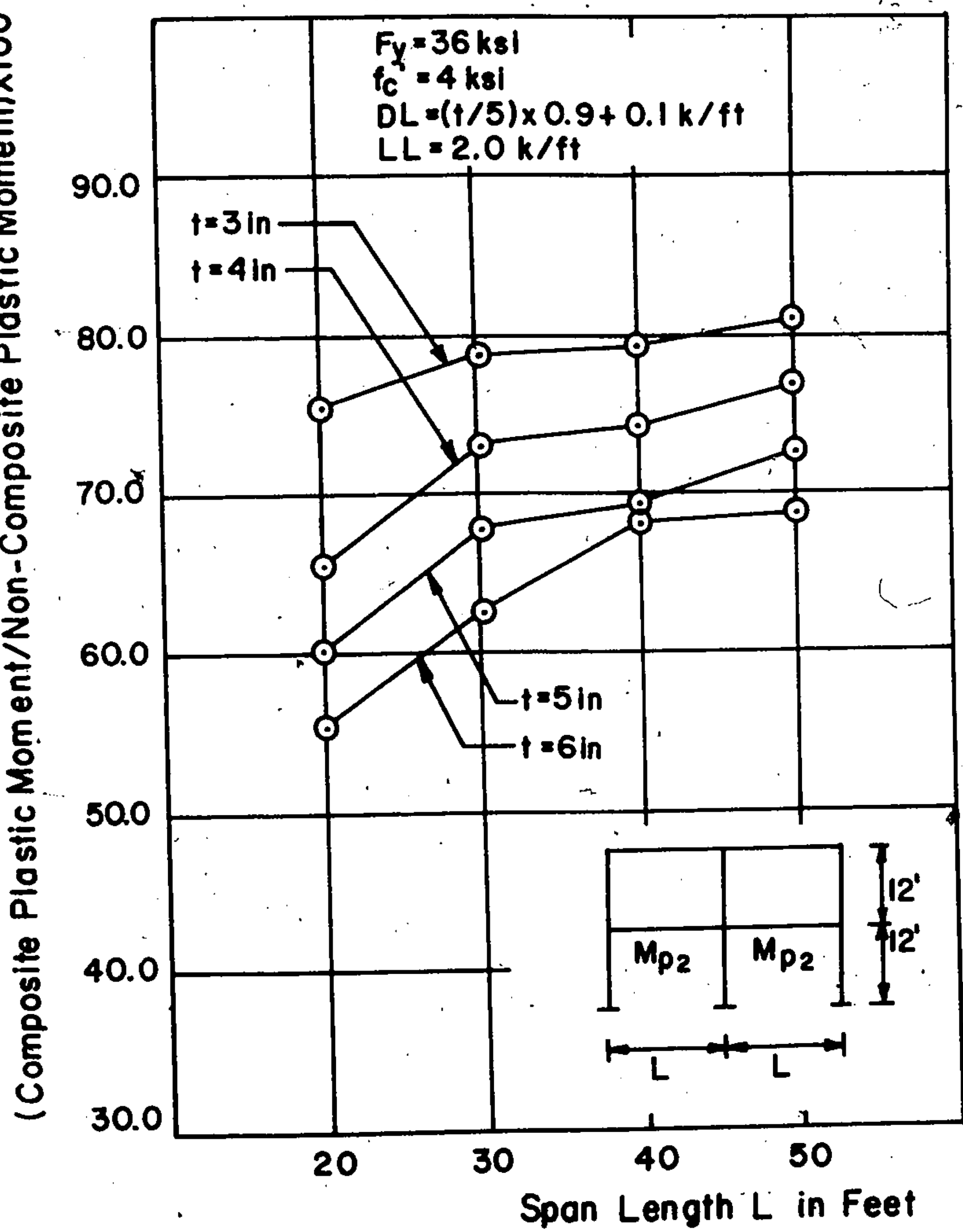


Fig. 4-8 Empirical Relationship between Composite Plastic Moment/Non-Composite Plastic Moment, Slab Thickness and Span Length for a Floor Beam of Two-Story Two-Bay Example Frame

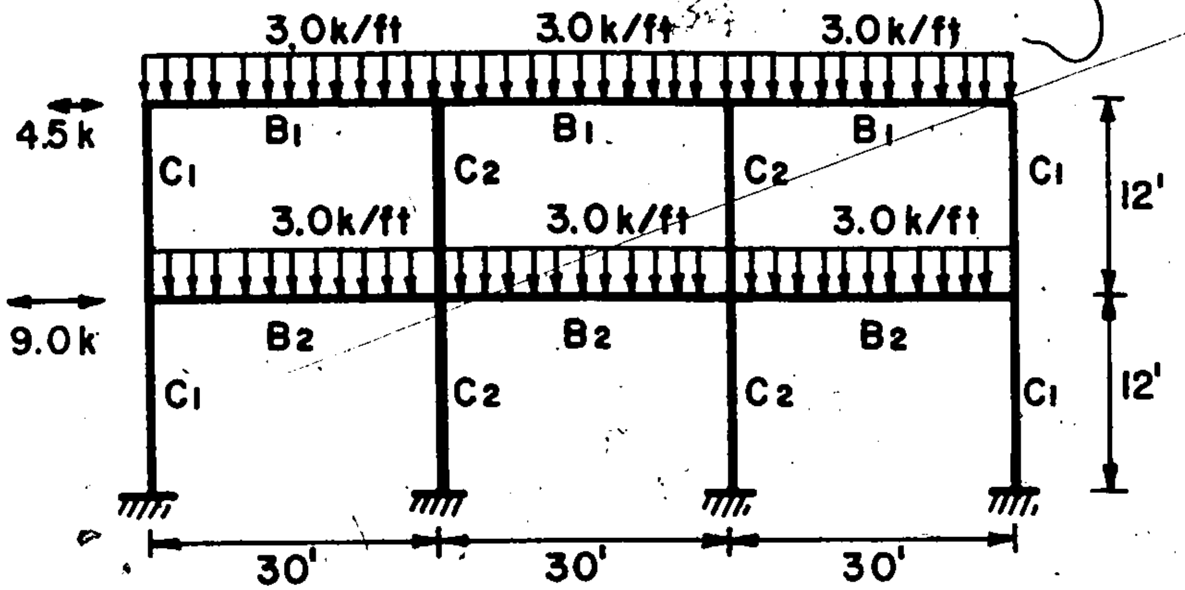
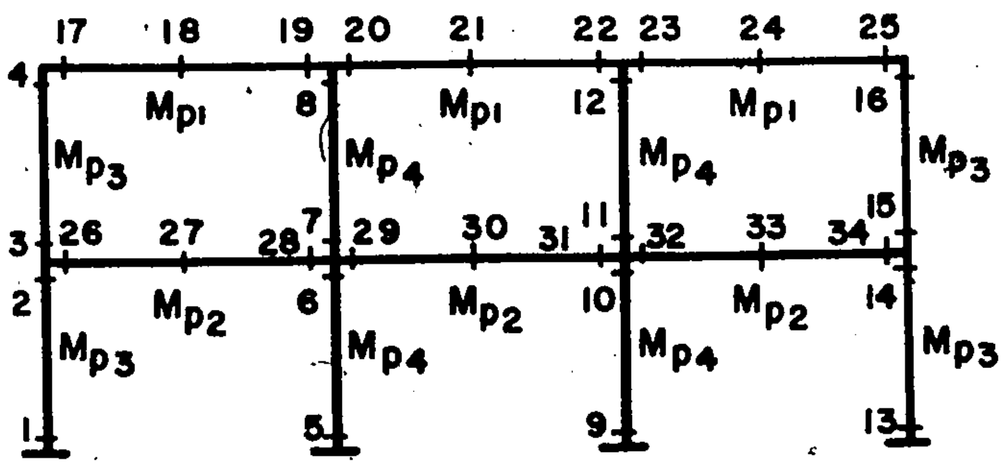
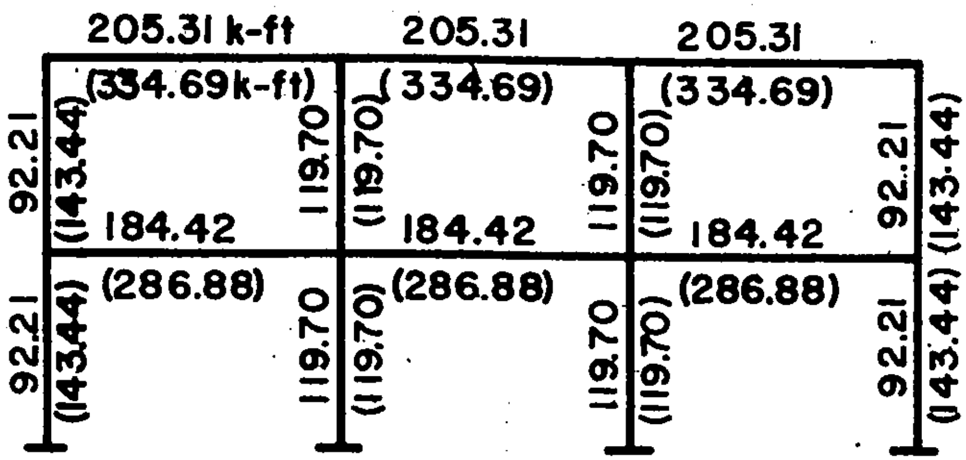


Fig. 5-1 Symmetrical Two-Story Three-Bay Composite Frame Showing Working Loads



Where $M_{p3} \geq 57.0$ k-ft and $M_{p4} \geq 119.7$ k-ft.

Fig. 5-2 Idealized Symmetrical Two-Story Three-Bay Composite Frame



Where ( ) shows linear programming solution for non-composite case.

Fig. 5-3 Linear Programming Solution for Two-Story Three-Bay Composite Frame

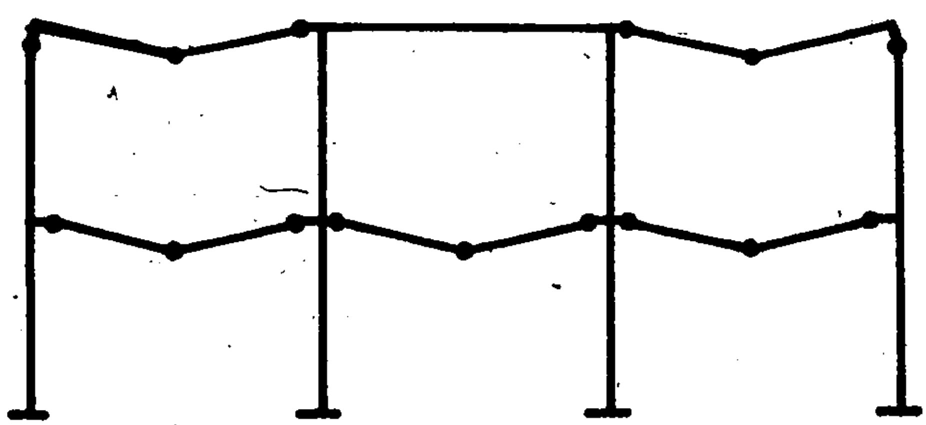
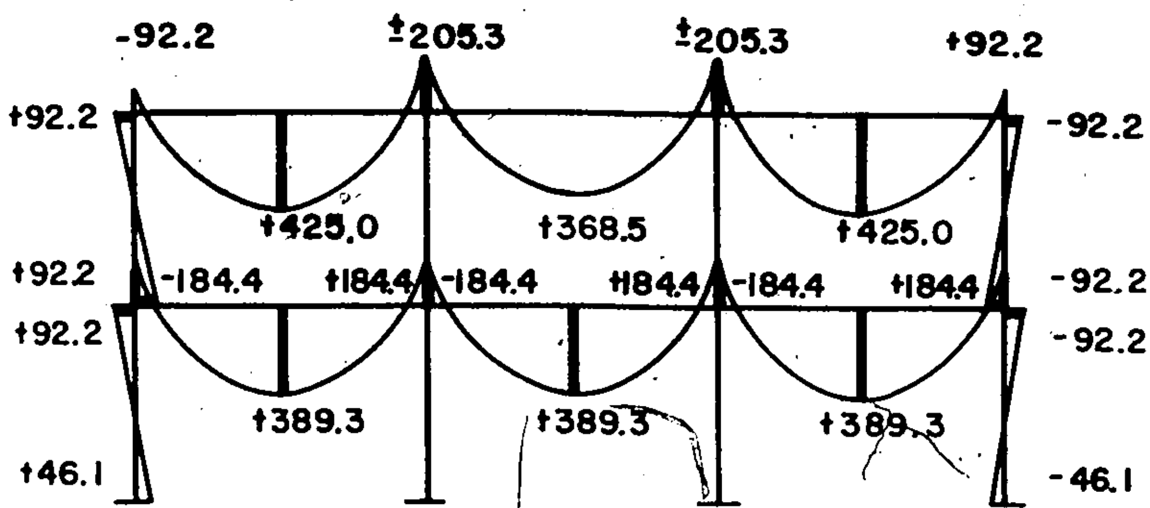
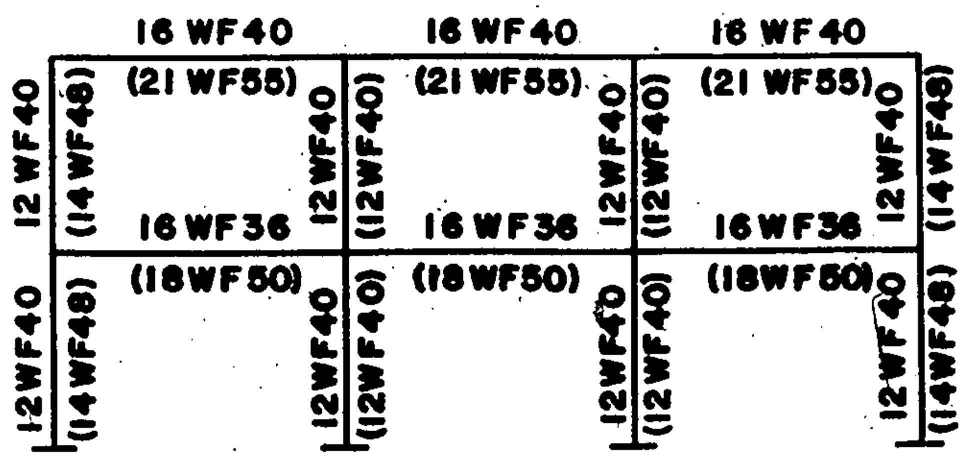


Fig. 5-4 Critical Collapse Mechanism for Symmetrical Two-Story Three-Bay Composite Frame



Where unit is k-ft.

Fig. 5-5 Moment Distributions at Critical Collapse of Symmetrical Two-Story Three-Bay Composite Frame



Where ( ) shows member layout for non-composite case.

Fig. 5-6 Member Layout for Two-Story Three-Bay Composite Frame

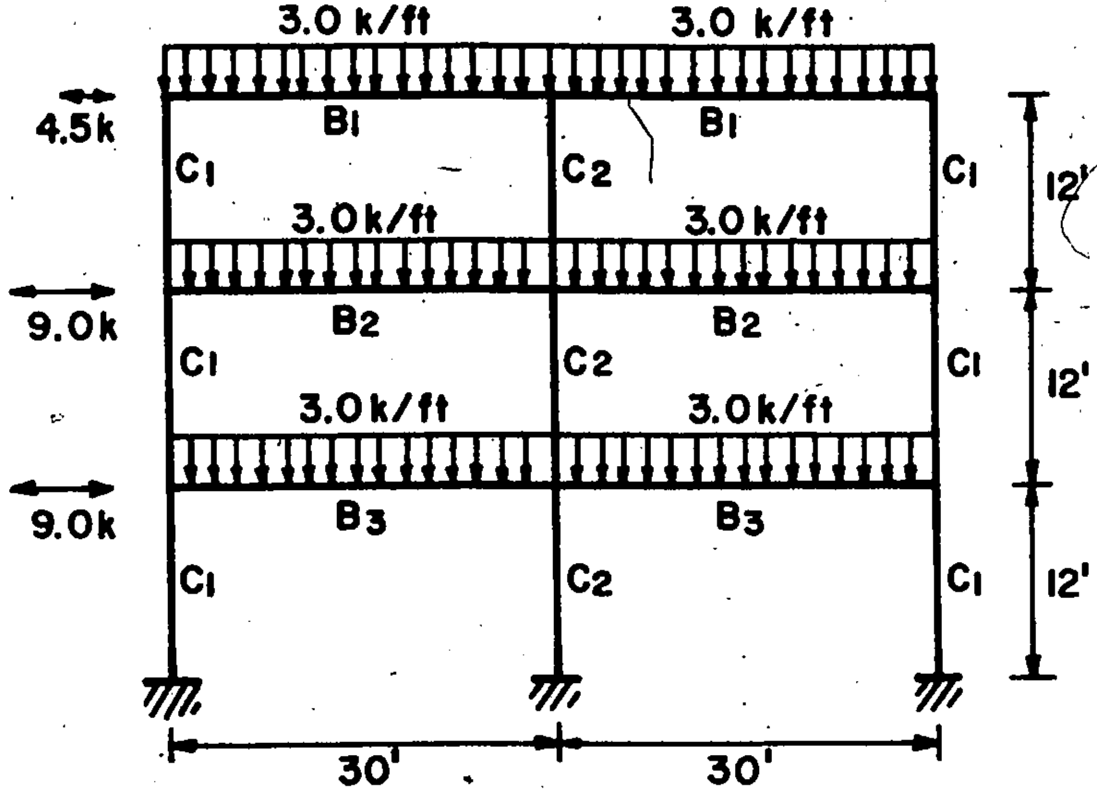
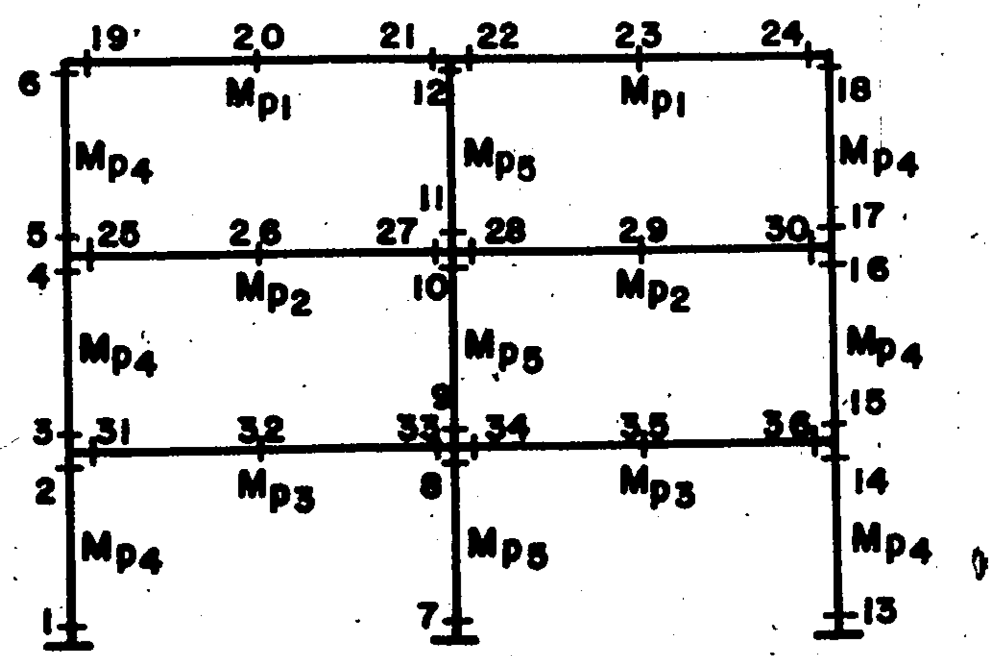
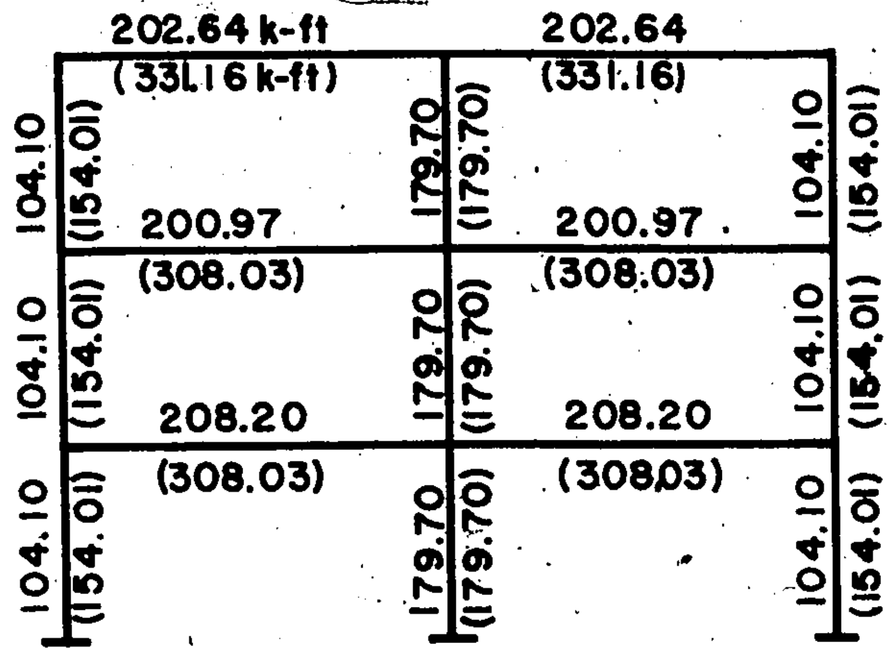


Fig. 5-7 Symmetrical Three-Story Two-Bay Composite Frame Showing Working Loads



Where $M_{p4} \geq 104.1$ and $M_{p5} \geq 179.7$ (k-ft).

Fig. 5-8 Idealized Symmetrical Three-Story Two-Bay Composite Frame



Where ( ) shows linear programming solution for non-composite case.

Fig. 5-9 Linear Programming Solution for Three-Story Two-Bay Composite Frame

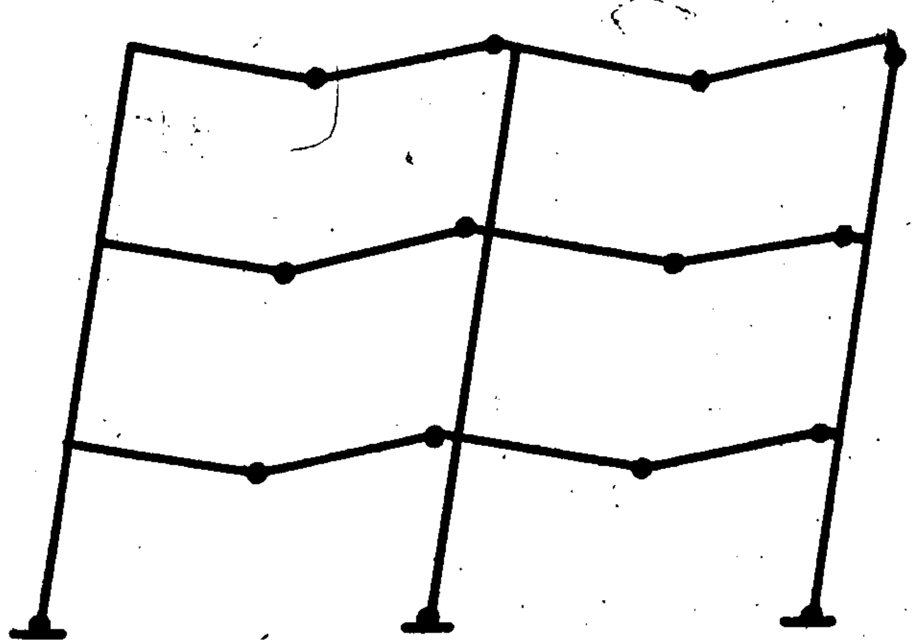
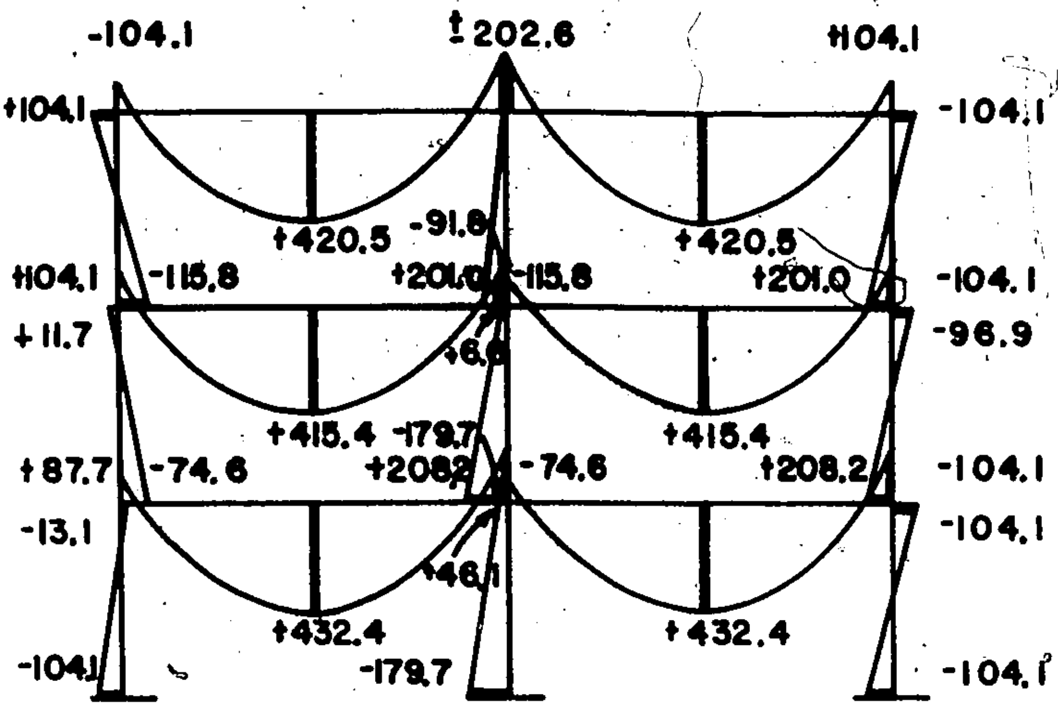
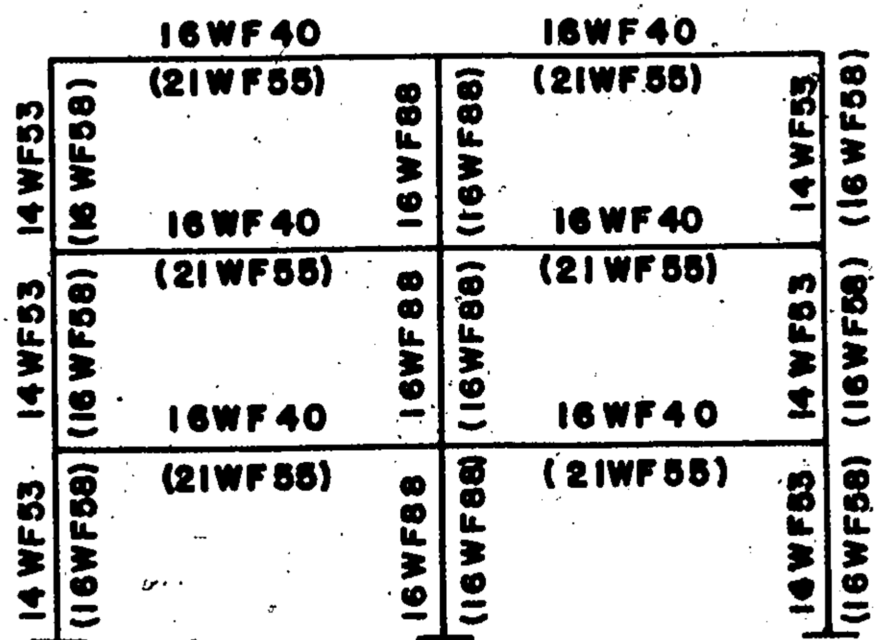


Fig. 5-10 Critical Collapse Mechanism for Symmetrical Three-Story Two-Bay Frame



Where unit is k-ft.

Fig. 5-11 Moment Distributions at Critical Collapse of Symmetrical Three-Story Two-Bay Composite Frame



Where ( ) shows member layout for non-composite case.

Fig. 5-12 Member Layout for Three-Story Two-Bay Composite Frame

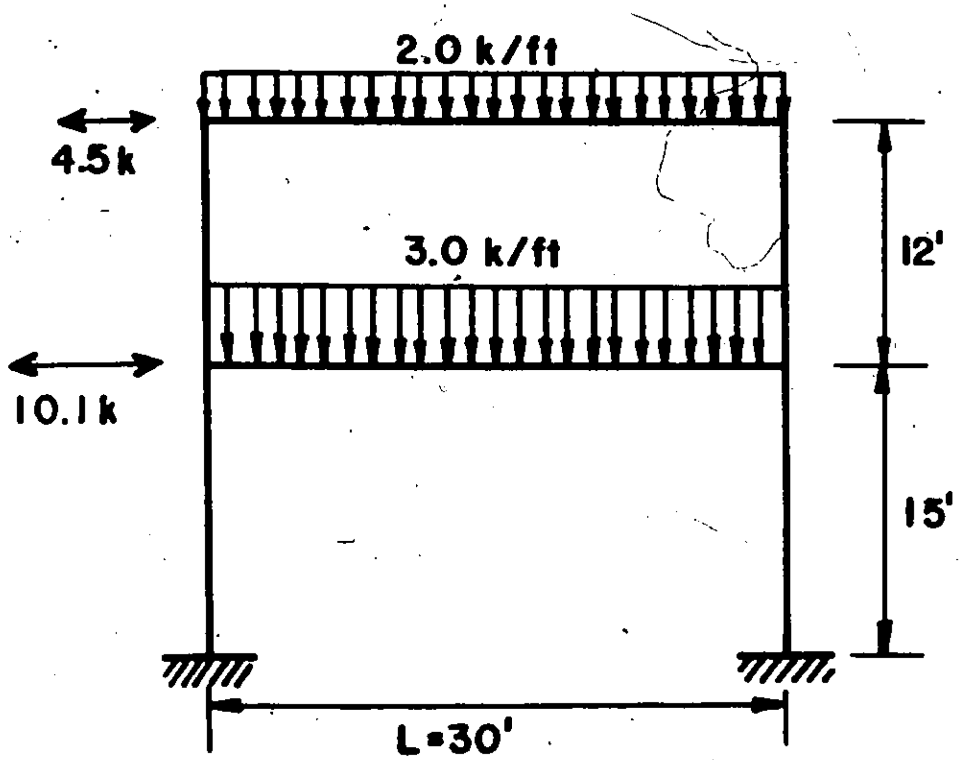


Fig. 6-1 Symmetrical Two-Story Single-Bay Steel Frame Showing Working Loads

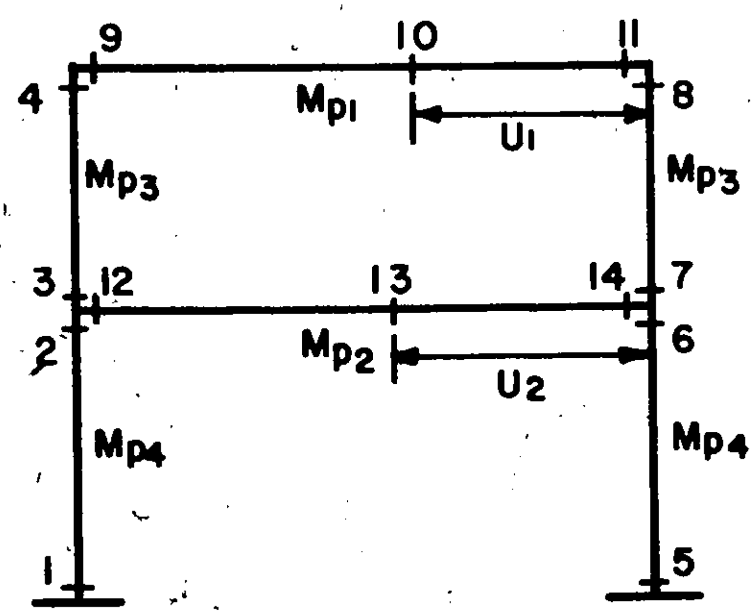
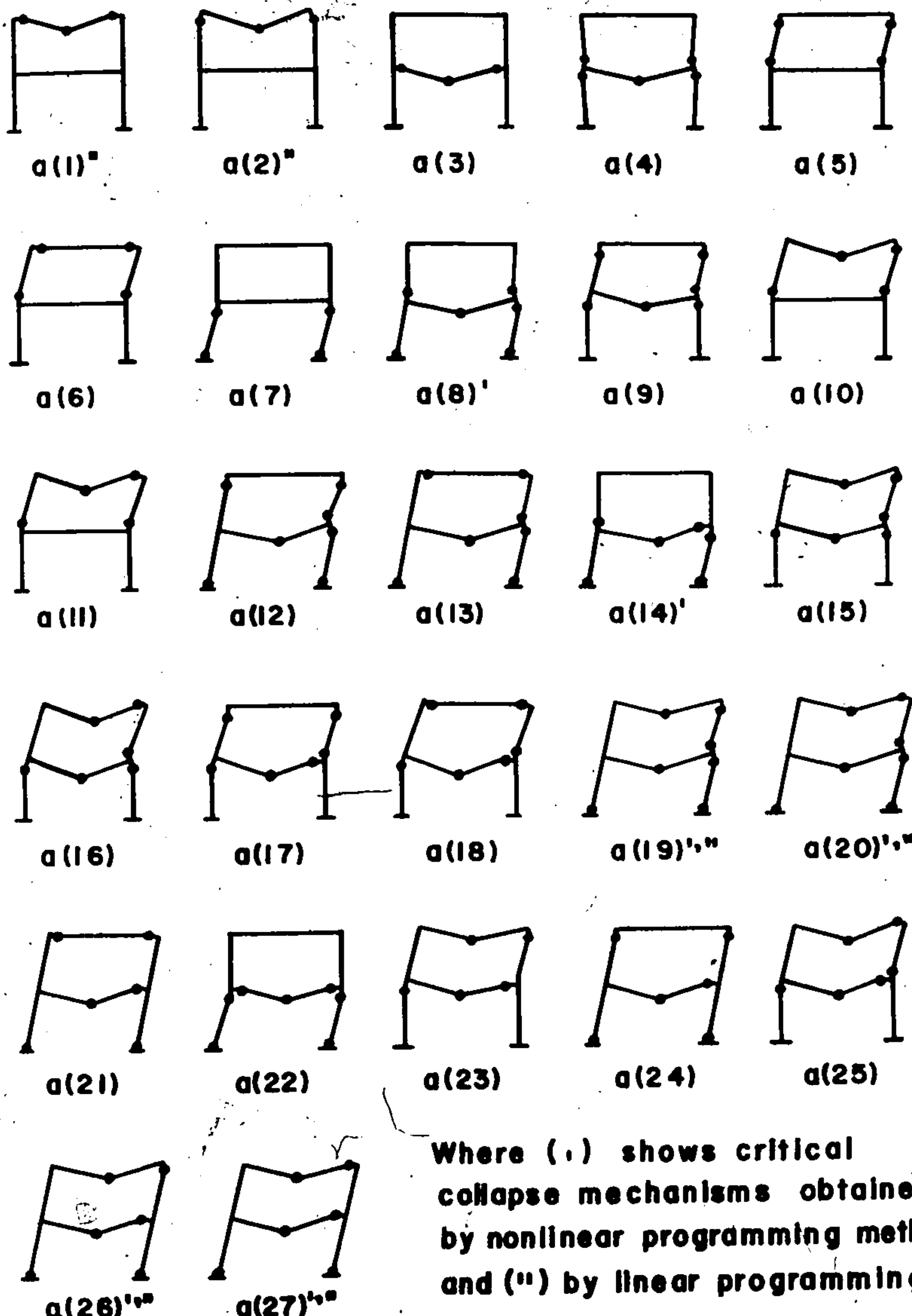


Fig. 6-2 Idealized Symmetrical Two-Story Single-Bay Steel Frame



Where (') shows critical collapse mechanisms obtained by nonlinear programming method and (") by linear programming method.

Fig. 6-3 Feasible Collapse Mechanisms for Symmetrical Two-Story Single-Bay Steel Frame

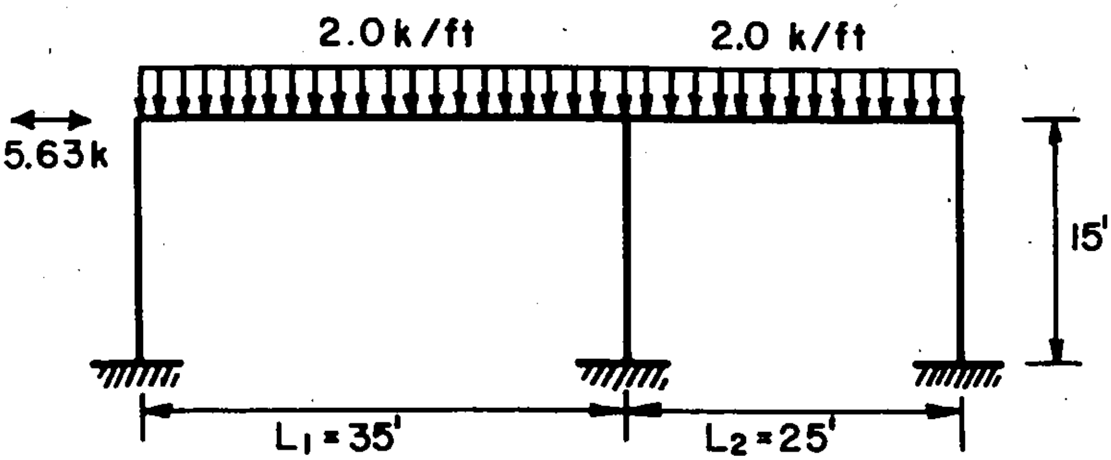


Fig. 6-4 Unsymmetrical One-Story Two-Bay Steel Frame Showing Working Loads

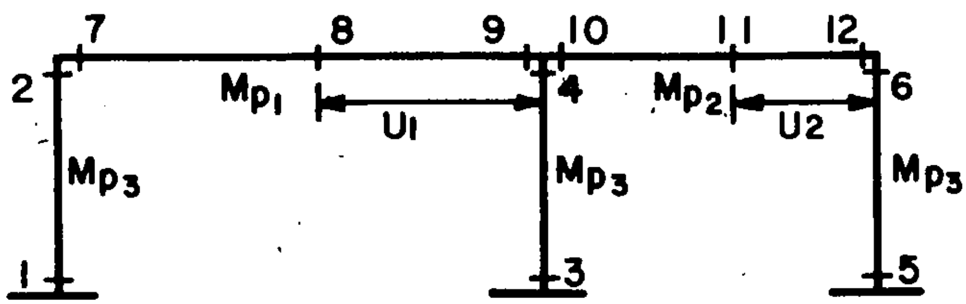
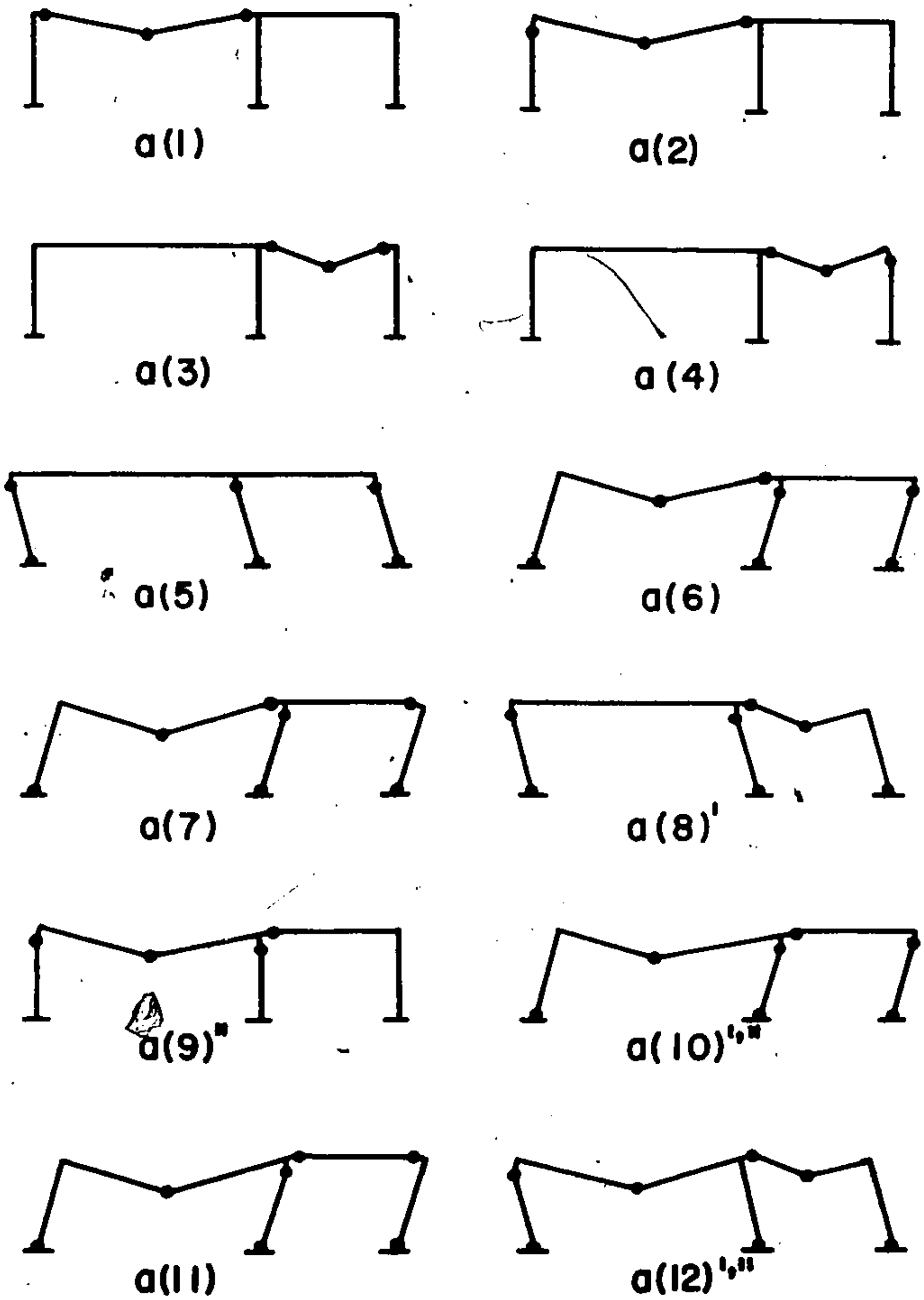


Fig. 6-5 Idealized Unsymmetrical One-Story Two-Bay Steel Frame



Where (') shows critical collapse mechanisms obtained by nonlinear programming method and ('') by linear programming method.

Fig. 6-6 Feasible Collapse Mechanisms for Unsymmetrical One-Story Two-Bay Steel Frame

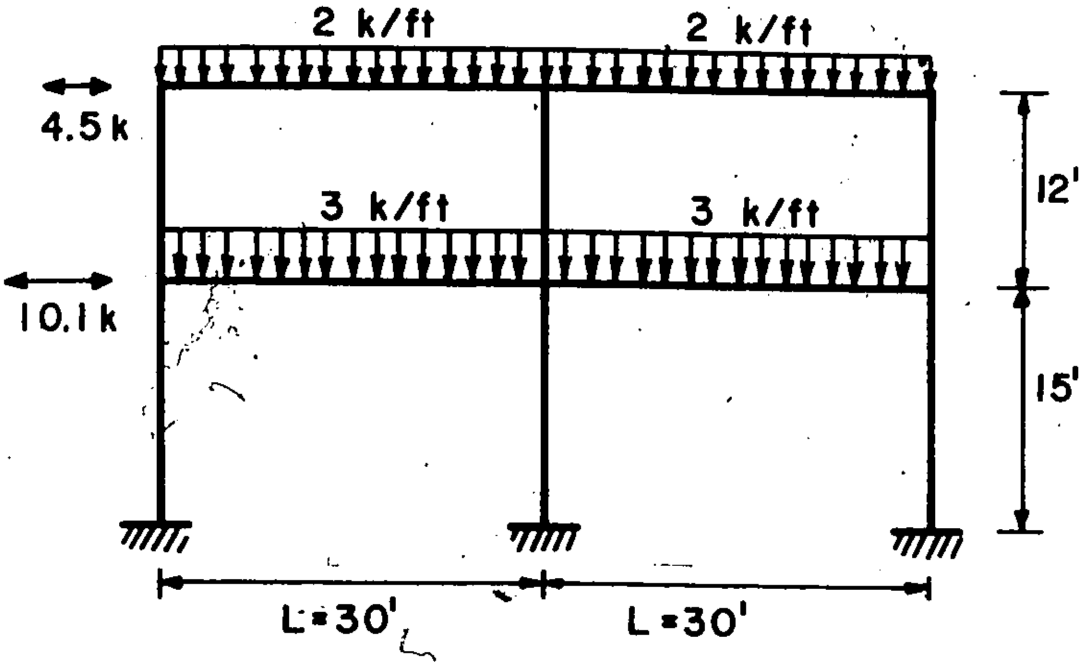


Fig. 6-7 Symmetrical Two-Story Two-Bay Composite Frame Showing Working Loads

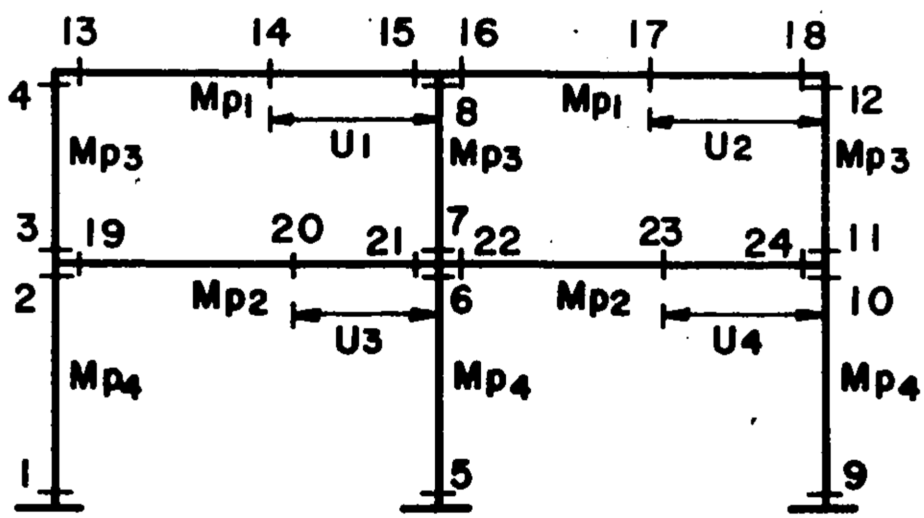


Fig. 6-8 Idealized Symmetrical Two-Story Two-Bay Composite Frame

a(3);

a(8);

a(11);

a(17);

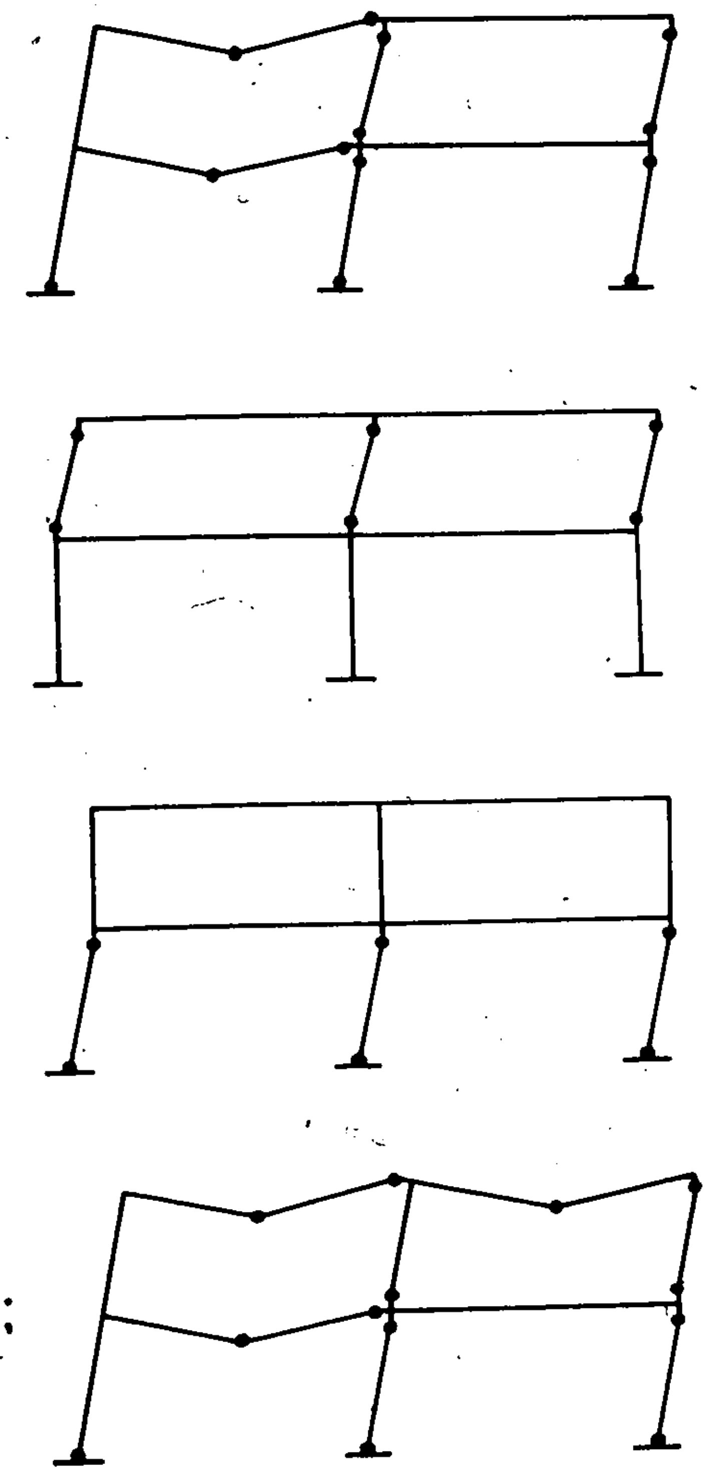


Fig. 6-9 Critical Collapse Mechanisms corresponding to Active Constraints for Symmetrical Two-Story Two-Bay Composite Frame

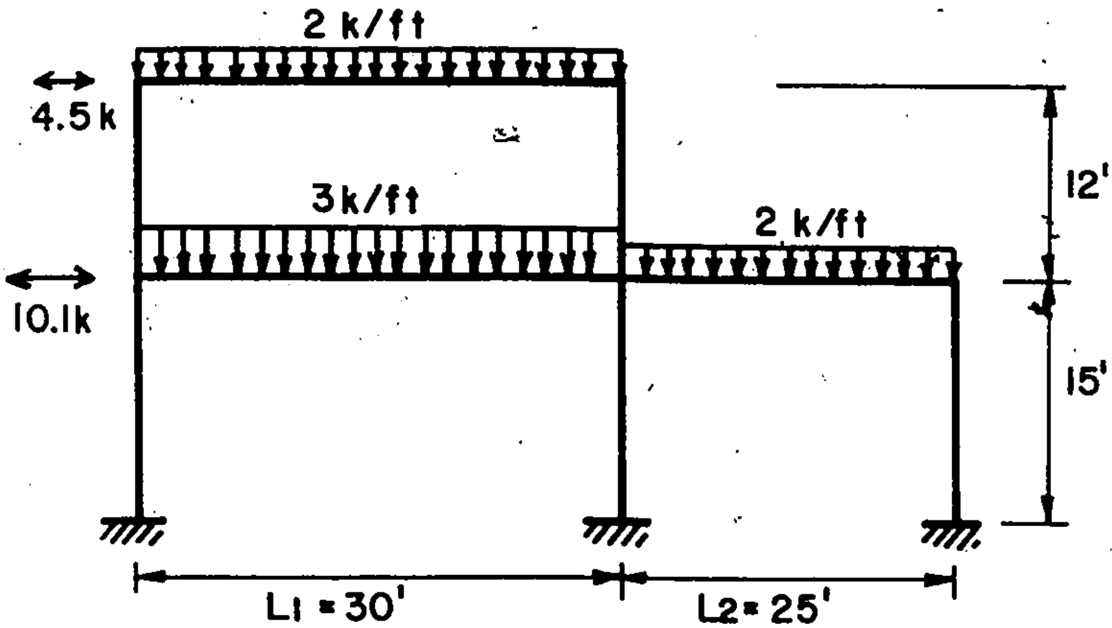


Fig. 6-10 Unsymmetrical Composite Frame Showing Working Loads

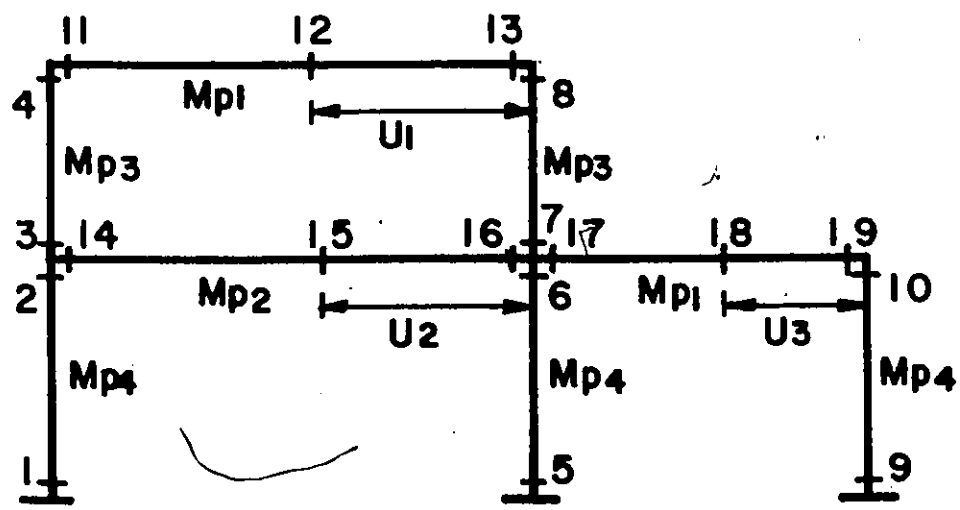


Fig. 6-11 Idealized Unsymmetrical Composite Frame

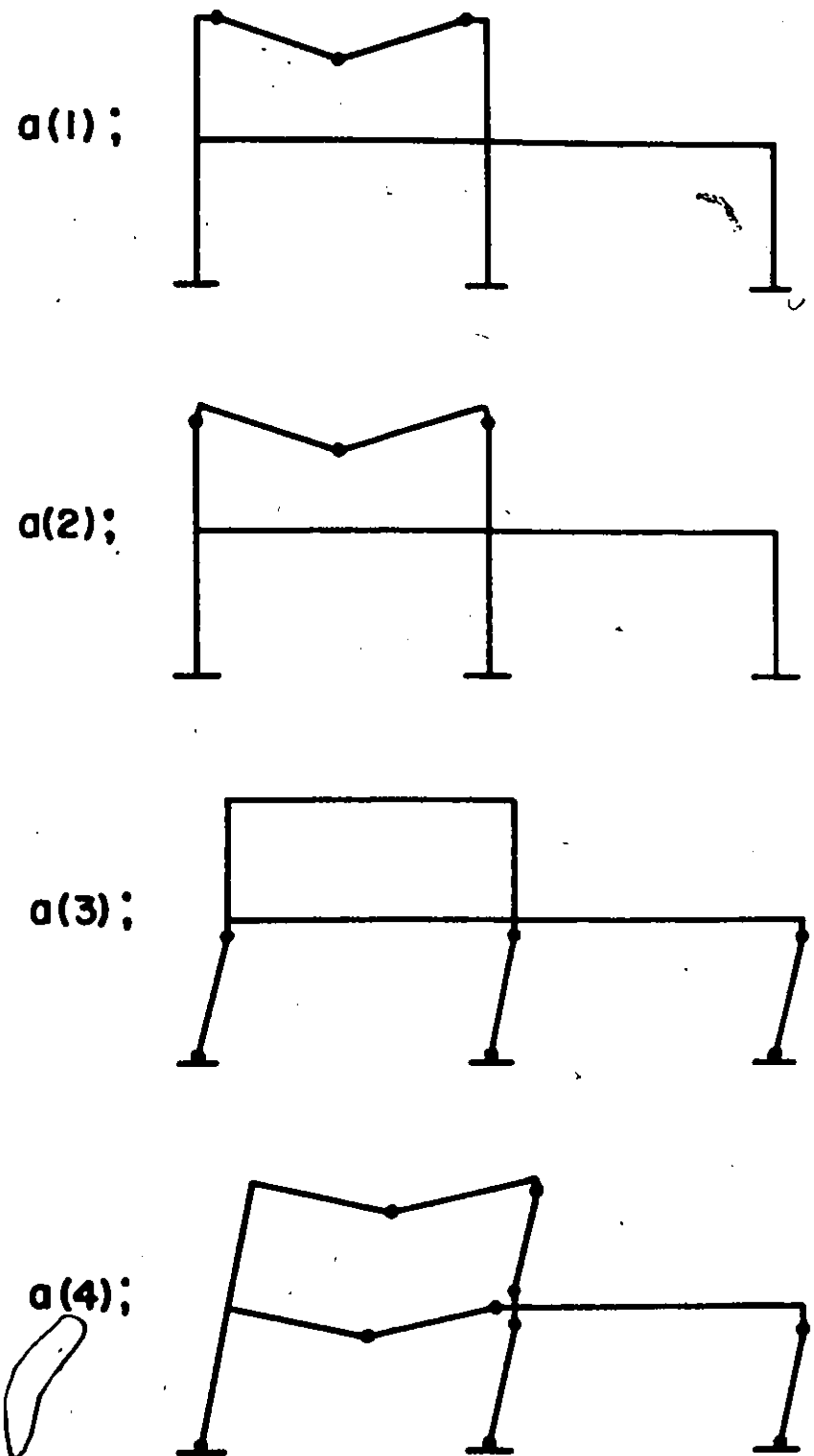


Fig. 6-12 Critical Collapse Mechanisms corresponding to Active Constraints for Unsymmetrical Composite Frame

APPENDIX 2

TABLES

TABLE 4-1

Available Economical Rolled Wide Flanges and Ultimate Moment Capacities for A36 Steel

No	Shapes	M_u (K-ft)	No	Shapes	M_u (K-ft)
1	8WF20	57.3	26	27WF94	834.0
2	8WF24	69.3	27	27WF102	912.0
3	8WF28	81.3	28	30WF99	936.0
4	10WF25	88.5	29	21WF127	954.0
5	12WF27	114.0	30	21WF142	1071.0
6	8WF40	119.7	31	30WF116	1134.0
7	12WF31	132.0	32	33WF118	1242.0
8	12WF36	154.2	33	24WF145	1248.0
9	14WF34	163.5	34	27WF145	1356.0
10	12WF45	194.7	35	33WF130	1398.0
11	16WF40	218.1	36	33WF141	1539.0
12	10WF60	225.3	37	33WF152	1674.0
13	16WF50	278.1	38	36WF150	1740.0
14	16WF58	318.0	39	36WF160	1869.0
15	18WF55	336.0	40	36WF170	2001.0
16	21WF55	375.0	41	36WF194	2301.0
17	18WF64	396.0	42	33WF220	2508.0
18	12WF99	456.0	43	36WF230	2829.0
19	21WF68	480.0	44	36WF245	3024.0
20	24WF68	595.0	45	36WF260	3228.0
21	24WF76	600.0	46	36WF280	3501.0
22	12WF133	630.0	47	36WF300	3765.0
23	24WF84	672.0			
24	27WF84	729.0			
25	12WF161	770.0			

TABLE 4-2

Approximate Functions for M_u/M_p (F_y = 36.0 Ksi and f_c' = 4.0 Ksi)

	Coeff. of Functions \ Thickness of Conc. Slab (in.)	3.0	4.0	5.0	6.0
Type A	a	0.5969	0.7644	0.8954	0.1006×10
	b	-0.4972×10^{-3}	-0.6357×10^{-3}	-0.6973×10^{-3}	-0.7395×10^{-3}
	c	0.2489×10^{-6}	0.3054×10^{-6}	0.3215×10^{-6}	0.3334×10^{-6}
	d	-0.3953×10^{-10}	-0.4679×10^{-10}	-0.4833×10^{-10}	-0.4955×10^{-10}
	Standard Deviation	0.076	0.087	0.105	0.130
Type B	a	0.6212×10	0.6404×10	0.7533×10	0.8740×10
	b	-0.1863×10	-0.1631×10	-0.1947×10	-0.2306×10
	c	0.2469	0.1863	0.2233	0.2664
	d	-0.1156×10^{-1}	-0.7665×10^{-2}	-0.9424×10^{-2}	-0.1139×10^{-1}
	Standard Deviation	0.043	0.042	0.051	0.071
	Mean (M_u/M_p)	1.475	1.639	1.804	1.969
	Standard Deviation	0.187	0.267	0.341	0.410
	Max (M_u/M_p)	1.997	2.346	2.680	3.005
	Min (M_u/M_p)	1.225	1.308	1.369	1.445

Where Type A; $M_u/M_p = e^{a + b M_p + c M_p^2 + d M_p^3}$ (if $M_p \approx 0$)

Type B; $M_u/M_p = a + b (\log M_p) + c (\log M_p)^2 + d (\log M_p)^3$ (if $M_p \neq 0$)

TABLE 4-3

Approximate Function for M_u/M_p (F_y = 36.0 Ksi and f_c' = 3.5 Ksi)

	Coeff. of Functions \ Thickness of Conc. Slab (in.)	3.0	4.0	5.0	6.0
Type A	a	0.5771	0.7531	0.8884	0.1001×10
	b	-0.4839×10^{-3}	-0.6505×10^{-3}	-0.7171×10^{-3}	-0.7572×10^{-3}
	c	0.2403×10^{-6}	0.3169×10^{-6}	0.3331×10^{-6}	0.3419×10^{-6}
	d	-0.3808×10^{-10}	-0.4888×10^{-10}	-0.5018×10^{-10}	-0.5084×10^{-10}
	Standard Deviation	0.077	0.089	0.104	0.128
Type B	a	0.6385×10	0.6366×10	0.7347×10	0.8608×10
	b	-0.1967×10	-0.1613×10	-0.1847×10	-0.2237×10
	c	0.2674	0.1811	0.2040	0.2531
	d	-0.1271×10^{-1}	-0.7251×10^{-2}	-0.8246×10^{-2}	-0.1061×10^{-1}
	Standard Deviation	0.043	0.044	0.049	0.067
	Mean (M_u/M_p)	1.451	1.615	1.780	1.946
	Standard Deviation	0.184	0.265	0.342	0.413
	Max (M_u/M_p)	1.971	2.326	2.664	2.992
	Min (M_u/M_p)	1.205	1.291	1.351	1.421

TABLE 4-4

Approximate Function for M_u/M_p (F_y = 36.0 Ksi and f_c' = 3.0 Ksi)

	Coeff. of Functions \ Thickness of Conc. Slab (in.)	3.0	4.0	5.0	6.0
Type A	a	0.5531	0.7366	0.8790	0.9948
	b	-0.4676×10^{-3}	-0.6630×10^{-3}	-0.7438×10^{-3}	-0.7831×10^{-3}
	c	0.2274×10^{-6}	0.3285×10^{-6}	0.3498×10^{-6}	0.3557×10^{-6}
	d	-0.3578×10^{-10}	-0.5118×10^{-10}	-0.5286×10^{-10}	-0.5295×10^{-10}
	Standard Deviation	0.079	0.091	0.105	0.125
Type B	a	0.6388×10	0.6710×10	0.7261×10	0.8370×10
	b	-0.2007×10	-0.1795×10	-0.1794×10	-0.2108×10
	c	0.2770	0.2090	0.1912	0.2288
	d	-0.1334×10^{-1}	-0.8611×10^{-2}	-0.7348×10^{-2}	-0.9169×10^{-2}
	Standard Deviation	0.043	0.045	0.048	0.063
	Mean (M_u/M_p)	1.423	1.587	1.751	1.916
	Standard Deviation	0.179	0.262	0.342	0.415
	Max (M_u/M_p)	1.937	2.300	2.642	2.974
	Min (M_u/M_p)	1.182	1.268	1.333	1.395

TABLE 4-5

Maximum Plastic Moment for a Symmetrical Two-Story Two-Bay Frame Example (w = 3 K/ft, LL/DL =2)

f'_c (Ksi)		M_p (K-ft) \ Span length (ft)	20.0	30.0	40.0	50.0
-	Non-Composite	M_{p_1}	155.83	219.39	579.70	914.39
		M_{p_2}	143.93	286.88	510.00	796.88
		M_{p_3}	71.97	189.34	300.90	444.34
		M_{p_4}	32.87	0.00	0.00	0.00
3.0	Composite	M_{p_1}	83.88 (53.8%)	215.90 (67.6%)	415.35 (71.6%)	657.90 (72.0%)
		M_{p_2}	88.10 (61.2%)	198.06 (69.0%)	362.34 (71.0%)	592.94 (74.4%)
		M_{p_3}	46.83 (65.1%)	99.03 (52.3%)	181.17 (60.2%)	342.37 (77.1%)
		M_{p_4}	44.05 (134%)	24.16(-----)	30.60 (-----)	0.00 (0.0%)
3.5	Composite	M_{p_1}	83.17 (53.4%)	213.10 (66.7%)	408.93 (70.5%)	647.21 (70.8%)
		M_{p_2}	87.19 (60.6%)	195.91 (68.3%)	357.54 (70.1%)	584.55 (73.4%)
		M_{p_3}	47.05 (65.4%)	97.95 (51.7%)	178.77 (59.1%)	338.17 (76.1%)
		M_{p_4}	43.60 (132.6%)	24.47 (----)	30.60 (----)	0.00 (0.0%)
4.0	Composite	M_{p_1}	82.64 (53.0%)	211.12 (66.1%)	404.23 (69.7%)	639.09 (69.9%)
		M_{p_2}	86.53 (60.1%)	194.39 (67.8%)	354.04 (69.4%)	578.22 (72.6%)
		M_{p_3}	47.22 (65.6%)	97.19 (51.3%)	177.[illegible]2 (58.8%)	335.01 (75.4%
		M_{p_4}	43.26 (131.6%)	24.69 (----)	30.60 (----)	0.00 (0.0%)

TABLE 6-1

Approximate Function for M_u - M_p Relation (F_y = 36.0 Ksi and f_c' = 4.0 Ksi)

Coeff. of Function \ Thickness of Conc. Slab (in)	3.0	4.0	5.0	6.0
a	0.3902×10^{2}	0.6186×10^{2}	0.7265×10^{2}	0.8971×10^{2}
b	0.1390×10	0.1519×10	0.1688×10	0.1850×10
c	-0.2970×10^{-4}	-0.7451×10^{-4}	-0.1348×10^{-3}	-0.1823×10^{-3}
d	-0.5020×10^{-8}	0.3258×10^{-8}	0.1148×10^{-7}	0.1803×10^{-7}
Standard Deviation	40.30	39.13	34.81	33.39
Mean (M_u)	1407.00	1515.82	1626.48	1746.13
Standard Deviation	1232.53	1300.01	1358.11	1433.07
Max (M_u)	4611.88	4923.90	5154.08	5441.07
Min (M_u)	114.41	134.43	153.56	172.21

Where Type of Function; $M_u = a + b\,M_p + c\,M_p^2 + d\,M_p^3$ (K-ft)

TABLE 6-2

Approximate Function for M_u - M_p Relation (F_y = 36.0 Ksi and f_c' = 3.5 Ksi)

Coeff. of Function \ Thickness of Conc. Slab (in)	3.0	4.0	5.0	6.0
a	0.3513×10^2	0.6377×10^2	0.7742×10^2	0.9074×10^2
b	0.1385×10	0.1477×10	0.1655×10	0.1823×10
c	-0.4202×10^{-4}	-0.5550×10^{-4}	-0.1283×10^{-3}	-0.1821×10^{-3}
d	-0.2677×10^{-8}	-0.7278×10^{-10}	0.1080×10^{-7}	0.1807×10^{-7}
Standard Deviation	38.56	48.36	34.89	33.28
Mean (M_u)	1384.72	1496.14	1602.99	1719.62
Standard Deviation	1211.67	1286.36	1339.36	1407.51
Max (M_u)	4535.45	4859.92	5087.97	5348.90
Min (M_u)	112.93	133.30	152.64	171.44

Where the units for M_u and standard deviations are K-ft.

TABLE 6-3

Approximate Function for M_u - M_p Relation (F_y = 36 Ksi and f_c' = 3.0 Ksi)

Coeff of Function \ Thickness of Conc. Slab (in)	3.0	4.0	5.0	6.0
a	0.3152×10^{2}	0.6389×10^{2}	0.8006×10^{2}	0.9324×10^{2}
b	0.1377×10	0.1435×10	0.1606×10	0.1781×10
c	-0.5726×10^{-4}	-0.3596×10^{-4}	-0.1134×10^{-3}	-0.1751×10^{-3}
d	0.5074×10^{-9}	-0.3950×10^{-8}	0.8952×10^{-8}	0.1729×10^{-7}
Standard Deviation	35.54	40.93	37.38	34.24
Mean (M_u)	1358.37	1471.54	1575.15	1687.84
Standard Deviation	1187.26	1267.69	1319.66	1379.73
Max (M_u)	4450.70	4772.92	5018.94	5250.99
Min (M_u)	110.97	131.79	151.41	170.41

Where the units for M_u and standard deviations are K-ft

APPENDIX 3

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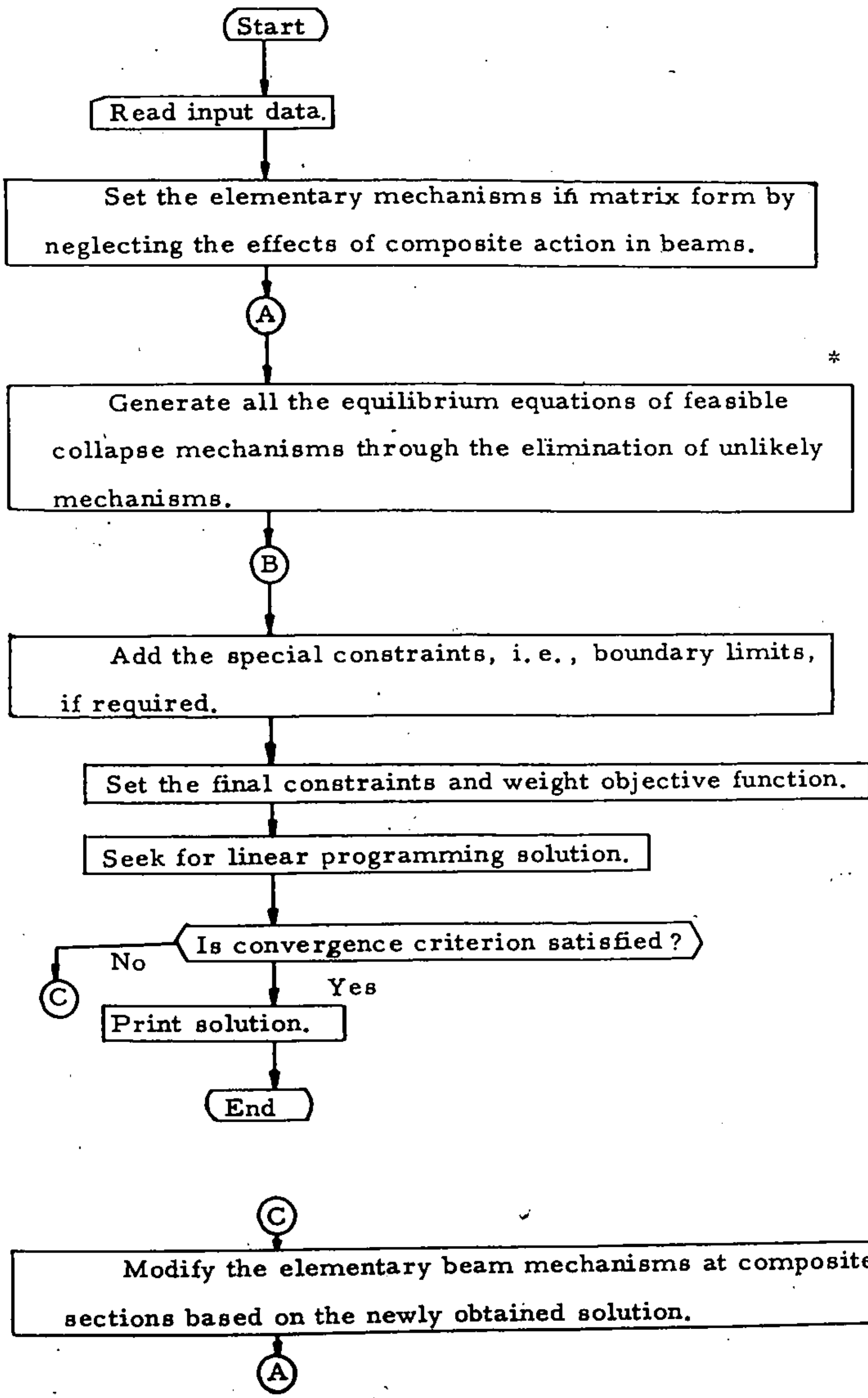
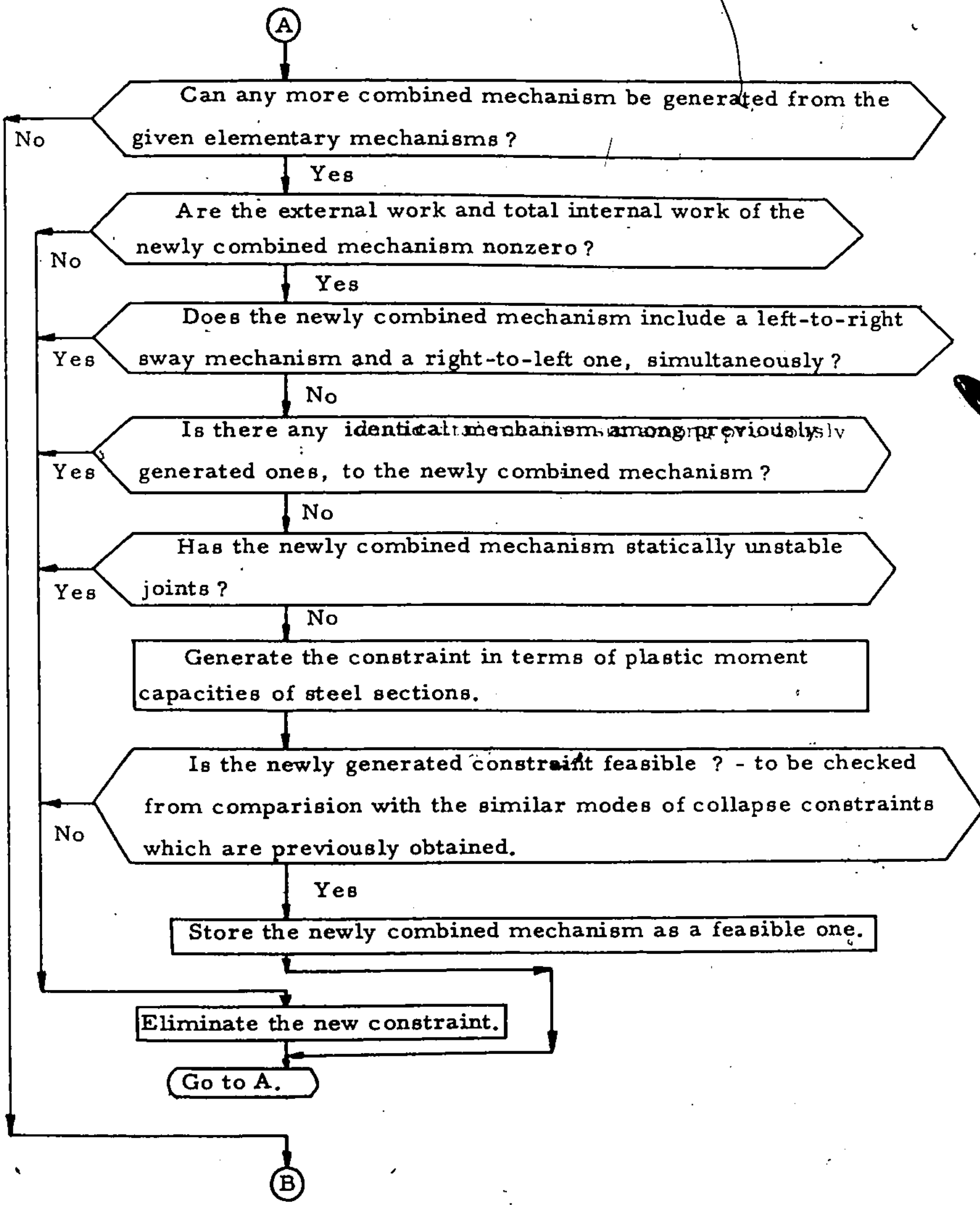
APPENDIX 4

DESCRIPTION AND LIST OF COMPUTER PROGRAMS

A. 4-1 Flow Chart of Main Program for Linear Optimization of Composite Steel Frames



(*) Detailed Block Chart for Part A to B



A. 4-2 Descriptions of Input Data for Linear Optimization of Composite Steel Frames

1) First Set of Data

NTELM, NOMP, NTHL, NBELM

Where NTELM, NOMP, NTHL and NBELM denote the number of all the elementary mechanisms, number of all the plastic moments defined as design variables, number of all the assumed critical sections, and number of the beam elementary mechanisms, respectively.

2) Second Groups of Data

NOH, WEXTN, WEXTD, MECO (i)

for i = 1, NTELM

Where NOH is the number of plastic hinges which occurs at the i-th elementary mechanism. WEXTN and WEXTD are the numerater and denominator of the coefficient of the corresponding external work WEXTO (i), respectively. MECO (i) is the i-th elementary mechanism's code number which is used in classifying the mechanism according to its way of collapse, and also to be used for eliminating the unlikely combined mechanisms composed of the right-to-left and left-to-right elementary sway mechanisms simultaneously. Assign integer number of -1, 0 and +1 to MECO (i) corresponding to right-to-left, non-lateral and left-to-right mechanisms if all of three types of mechanisms have to be considered, i.e., in the case of the unsymmetrical structures or loadings, otherwise leave blank. These input datas for elementary mechanisms have to be read in the order of beam and sway or joint or other mechanisms in order to identify beam mechanisms from another types of elementary mechanisms.

3) Third Groups of Data

J, MP, THTAN, THTAD

for i = 1, NTELM and k = 1, NOH

Where J means the J-th section at which the plastic hinge is assumed for the i -th elementary mechanism, and it must be given in negative integer if the corresponding hinge is assumed at composite section, otherwise given in positive integer. MP means the MP-th plastic moment which corresponds to the J-th section. THTAN and THTAD are the numerater and denominator for the value of the hinge rotation at the J-th section. These data represent the i-th elementary mechanism at J-th section.

4) Fourth Set of Data

JNT (j), j = 1, NTHL

Where JNT (j) is assigned to the smallest integer number among those assigned to the critical sections around a joint of near the j -th section, including the number j. In addition, JNT (j) is the j-th section's code number to be used for eliminating the unlikely combined mechanisms with the elementary joint mechanism separately near the j-th section. If the number of the assumed critical sections near the j-section is only one, i.e., at supports or mid-span, JNT (j) must be identical to a negative integer -j for a convenience in the elimination procedures.

5) Fifth Set of Data

MINHL, MXNHL, MINCM, IPRT, NP

Where MINHL, MXNHL and MINCM denote the minimum number of the required hinges for all through the feasible mechanisms, maximum number of the required hinges for and the minimum number of the combined feasible mechanisms to be generated, respectively. MINHL, MXNHL and MINCM, however, may be assumed only for obtaining an approximate solution, otherwise for the generation of all feasible mechanisms must be left blank.

If the detailed process during optimization by computer is to be known, IPRT must be given in any positive integer, otherwise left blank. NP is the total number of the linear programming solutions of the original one and additional particular ones in which some of the original variables are considered to depend on the rest ones, and it results in the certain number of the different combinations of the particular variables. If such a particular solution is not needed as usual, NP is equal to 1.

6) Six Set of Data

FLENG, WL, WD, T

Where FLENG, WL and WD denote the standard span length (ft) for the proposed frame, the standard working live load and dead load acting on the typical beam including its own weight (K/ft), respectively. T denotes the thickness of the concrete slab (inch).

7) Seventh Set of Data

PMMIN (MP), MP = 1, NOMP

Where PMMIN (MP) denotes the value of lower limit (K-ft) for the MP-th variable.

8) Eighth Groups of Data

CO (ℓ, k), ℓ = 1, . . . , 4

. . . for k = 1, 2

Where CO (ℓ, 1) and CO (ℓ, 2), ℓ = 1, . . . , 4 denote the coefficients of a, b, c and d for the type A and the type B functions representing the continuous variations of M_u/M_p, respectively. (See Tables, 4-2 through 4-4)

9) Ninth and Last Groups of Data

NMP (j), j = 1, NOMP

... for k = 1, NP

Where NMP (j) denotes the new variable number corresponding to the j-th original variable to be used for obtaining the k-th particular solution. If NP is equal to 1, these data are not required.

A. 4-3 List of Programs for Linear Optimization of Composite Steel Frames

```
//UOCCC4C7      JOB        (
//                ),'ARAC                          ',MSGLEVEL=(1,1),
//                CLASS=C,TYPRUN=HOLD
//        EXEC FORTGCLG,REGION.FORT=150K,REGION.GO=440K,TIME.GO=99
//FORT.SYSIN DD *
IEF373I STEP /FORT     / START 72301.2304
IEF374I STEP /FORT     / STOP  72301.2305 CPU   0MIN 32.89SEC MAIN 15?K LCS   0K
IEF373I STEP /LKED     / START 72301.2305
IEF374I STEP /LKED     / STOP  72301.2305 CPU   0MIN 01.75SEC MAIN 100K LCS   0K
//GO.SYSIN DD *
IEF373I STEP /GO       / START 72301.2305
IEF374I STEP /GO       / STOP  72302.0003 CPU  52MIN 28.79SEC MAIN 4?4K LCS   0K
IEF375I  JOB /UOCC0407/ START 72301.2304
IEF376I  JOB /UOCC0407/ STOP  72302.0003 CPU  53MIN 10.43SEC
```

```
FORTRAN IV G LEVEL  20                  MAIN               DATE = 72301

*OPTIONS IN EFFECT*  ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,MAP
*OPTIONS IN EFFECT*  NAME = MAIN     , LINECNT =       45
*STATISTICS*    SOURCE STATEMENTS =        641,PROGRAM SIZE =   404514
*STATISTICS*  NO DIAGNOSTICS GENERATED
```

```
FORTRAN IV G LEVEL  20                  LINEAR             DATE = 72301

*OPTIONS IN EFFECT*  ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,MAP
*OPTIONS IN EFFECT*  NAME = LINEAR   , LINECNT =       45
*STATISTICS*    SOURCE STATEMENTS =        126,PROGRAM SIZE =     48?C
*STATISTICS*  NO DIAGNOSTICS GENERATED

*STATISTICS*  NO DIAGNOSTICS THIS STEP
```

```
      C      ...
      C      BY LIN...
      C      WRITTEN BY S. ARAC , DEPARTMENT OF CIVIL ENGINEERING
      C      UNIVERSITY OF OTTAWA   OTTAWA   CANADA
      C
      C      LIMITATIONS FOR THE PROGRAM ARE THE FOLLOWINGS
      C      THE MAXIMUM NUMBER OF HINGE LOCATIONS IS LIMITED TO 36
      C      THE MAXIMUM NUMBER OF PLASTIC MOMENTS APPLICABLE IS 10
      C      THE MAXIMUM NUMBER OF COMBINED MECHANISMS IS 800
      C      THE MAXIMUM NUMBER OF EACH COMBINED MECHANISMS IS 400
      C      THE MAXIMUM NUMBER OF ELEMENTARY MECHANISMS IS 30
      C
             DOUBLE PRECISION THETA(400,36,2),A(10000),WNT(800,10)
             DIMENSION THETAE(30,36),THETA1(36),WEXT(800),WXT(800),WX(400,2)
             DIMENSION WINT(800,10),NO(400),WTFF(10),WTF(10),FL(10)
             DIMENSION MPT(36),JNT(36),MECE(30),MEC(400,2),MELM(400,30,2)
             DIMENSION NMP(10),NREC(10),PM(10,2),IPM(10),IPMM(10),PMMIN(10)
             DIMENSION THETA0(30,36),WEXTO(30),MECO(30),ALP(36),MPT1(36)
             DIMENSION GBEN(2),CC(4,2)
             EQUIVALENCE (THETA(1),WNT(1)),(THETA(8001),A(1))
      C
             NREAD=1
             NWRIT=3
             MXELM=800
             MXMELM=400
             MXHL=36
             MXMP=10
             DO 5 J=1,MXHL
             MPT(J)=0.0
           5 CONTINUE
      C
      C      READ IN THE NUMBER OF THE TOTAL ELEMENTARY MECHANISMS
      C      READ IN THE NUMBER OF THE TOTAL PLASTIC MOMENTS
      C      READ IN THE NUMBER OF THE TOTAL HINGE LOCATIONS
      C      READ IN THE NUMBER OF THE BEAM ELEMENTARY MECHANISMS
      C
             READ(NREAD,10) NTELM,NOMP,NTHL,NBELM
          10 FORMAT(20I4)
          15 FORMAT(8F10.0)
      C
      C      ASSIGN THE INITIAL VALUES TO THE ELEMENTARY HINGE ROTATION MATRIX
      C
```

```
      C  J  ...
         (R J.., ...L
   18 THETA(I,J)=.
C
C     READ IN THE PROPERTIES OF EACH ELEMENTARY MECHANISMS
C     AND ARRANGE ALL ELEMENTARY MECHANISMS IN THE FOLLOWING ORDER
C     -BEAM,SWAY OR JOINT OR OTHER MECHANISMS
C
      MXNCM=NTELM
      DO 49 I=1,NTELM
      WRITE(NWRIT,20) I
   20 FORMAT (//'  THIS IS ELEMENTARY MECHANISM NO. =',I4/)
C
C     PUNCH A NEGATIVE,ZERO AND POSITIVE INTEGER FOR THE MECHANISM CODE
C     'MECE(I)' CORRESPONDING TO NEGATIVE,NON-HORIZONTAL AND POSITIVE
C     HORIZONTALLY DISPLACING MECHANISMS RESPECTIVELY,REQU,D FOR ELIMI.
C
      READ(NREAD,22) NOH,WEXTN,WEXTD,MECO(I)
   22 FORMAT(I4,2F10.4,I4)
      IF (WEXTD) 24,23,24
   23 WEXTD=1.0
   24 WEXTO(I)=WEXTN/WEXTD
      IF (MECO(I)) 25,27,26
   25 MECO(I)=-1
C
C     FIX THE NUMBER OF ACTUAL ELEMENTARY MECHANISMS
C     EXCLUDING THE DUPLICATED LATERAL MECHANISMS
C
      MXNCM=MXNCM-1
      GO TO 30
   26 MECO(I)=1
   27 IF (I-NBELM) 23,28,30
   28 WRITE(NWRIT,29) NOH,WEXTO(I),MECO(I)
   29 FORMAT('  THIS MECHANISM HAS',I4,'  HINGES   EXTERNAL WORK =',
     1F10.4,'(SF*W*L2)  MECHANISM CODE =',I3/)
      GO TO 32
   30 WRITE(NWRIT,31) NOH,WEXTO(I),MECO(I)
   31 FORMAT('  THIS MECHANISM HAS',I4,'  HINGES   EXTERNAL WORK =',
     1F10.4,'(SF)        MECHANISM CODE =',I3/)
   32 DO 49 K=1,NOH
C
C     PUNCH A NEGATIVE SIGN ON THE SECTION NUMBER 'J' WHICH CORRESPONDS
C     TO THE COMPOSITE SECTION
```

```
      READ(NREAD,33) J,THTAN,THTAD
   33 FORMAT(2I4,2F4.4)
      JJ=J
      IF (J) 34,35,35
   34 J=-J
   35 IF (THTAD) 38,37,38
   37 THTAD=1.
   38 IF (MXHL-J) 39,41,41
   39 WRITE(NWRIT,40) J
   40 FORMAT(1H1//'  THE MAXIMUM NUMBER OF HINGE LOCATIONS APPLICABLE IS
     1 EXCEEDED BY NO',I3)
      GO TO 9999
   41 IF (MXMP-MP) 42,44,44
   42 WRITE(NWRIT,43) MP
   43 FORMAT(1H1//'  THE MAXIMUM NUMBER OF PLASTIC MOMENTS APPLICABLE IS
     1 EXEEDED BY MP=',I3)
      GO TO 9999
   44 THETAD(I,J)=THTAN/THTAD
      MPT1(J)=JJ
      MPT(J)=MP
      IF (JJ) 47,45,45
   45 WRITE(NWRIT,46) J,THETAD(I,J),MPT(J)
   46 FORMAT('  THIS HINGE IS AT SECTION',I4,'   ANGLE ROTATION =',F9.4,
     1'AND PLASTIC MOMENT CAPACITY IS MP',I2)
      GO TO 49
   47 WRITE(NWRIT,48) J,THETAD(I,J),MPT(J)
   48 FORMAT('  THIS HINGE IS AT SECTION',I4,'   ANGLE ROTATION =',F9.4,
     1'AND PLASTIC MOMENT CAPACITY IS MP',I2,'  (COMP. SECTION)')
   49 CONTINUE
      WRITE(NWRIT,50)
   50 FORMAT(//10X,'WHERE THE NEGATIVE MECHANISM CODE NUMBER SHOWS A RIG
     1HT-TO-LEFT LATERAL MECHANISMS'/)
C
C     READ IN THE JOINT DATA FOR ELIMINATING MECHANISMS WITH STATICALLY
C     UNSTABLE JOINTS OR INDEPENDENT JOINT MECHANISMS
C
      READ(NREAD,10) (JNT(J),J=1,NTHL)
      DO 54 J=1,NTHL
      IF (JNT(J)) 54,53,54
   53 JNT(J)=J
   54 CONTINUE
C
```

```
      DO 58 J=1,NTHL
      IF (JNT(J)-J) 58,55,58
   55 J1=J+1
      IF (NTHL-J1) 58,56,56
   56 DO 57 JJ=J1,NTHL
      IF (JNT(J)-JNT(JJ)) 57,58,57
   57 CONTINUE
      JNT(J)=-JNT(J)
   58 CONTINUE
      IF (JNT(NTHL)-NTHL) 60,59,60
   59 JNT(NTHL)=-JNT(NTHL)
   60 WRITE(NWRIT,61)
   61 FORMAT(1H1/10X,'JOINT DATA'//5X,'JOINT NO.-CODE'/)
      DO 62 J=1,NTHL
   62 WRITE(NWRIT,63) J,JNT(J)
   63 FORMAT(5X,I9,'-',I4)
   64 FORMAT(8F10.4)
   65 FORMAT('  THIS IS MP',I2)
   68 FORMAT(/'  NO.',I4,'  (NO.',I4,')'/14(F9.3))
      WRITE(NWRIT,69)
   69 FORMAT(//10X,'WHERE THE NEGATIVE JOINT CODE NUMBER SHOWS A SINGLE
     1JOINT')
C
C     READ IN THE MINIMUM NUMBER OF THE REQUIRED HINGES
C     READ IN THE MAXIMUM NUMBER OF THE POSSIBLE HINGES FOR MECHANISMS
C     READ IN THE MINIMUM NUMBER OF THE CONSIDERABLE COMBINED MECHANISMS
C     READ IN A POSITIVE VALUE FOR 'IPRT' IF THE NUMBER OF ORIGINAL
C     MECHANISMS FOR COMBINED ONE NEEDED
C     READ IN THE NUMBER OF THE SOLUTIONS REQUIRED
C
      READ (NREAD,10) MINHL,MAXNHL,MINCM,IPRT,NP
C
C     READ IN STANDARD SPAN LENGTH,WORKING LOAD AND SLAB THICKNESS
C
      READ (NREAD,13) FLENG,WL,WS,T
      WT=WD+WL
C
C     READ IN THE COEFFICIENTS OF THE OBJECTIVE FUNCTION
C
      READ(NREAD,13) (WTFF(MP),MP=1,NOMP)
```

```
C     READ I[illegible]
      READ(NREAD,13) (PMMIN(MP),MP=1,NOMP)
C
C     READ IN THE COEFFICIENTS (C1,C2,C3 & C4) FOR THE FUNCTION MU/MP
C
      DO 70 K=1,2
   70 READ(NREAD,13) (CO(L,K),L=1,4)
      WRITE(NWRIT,71)
   71 FORMAT(1H1//20X,'APPROXIMATED FUNCTION FOR MU/MP'//
     110X,'CASE 1 : MU/MP=EXP(C1+C2*MP+C3*MP**2+C4*MP**3)',10X,
     2'CASE 2 : MU/MP=C1+C2*LOGMP+C3*(LOGMP)**2+C4*(LOGMP)**3'//)
      DO 72 L=1,4
   72 WRITE(NWRIT,73) L,CO(L,1),L,CO(L,2)
   73 FORMAT(16X,'C',I2,'=',F10.6,30X,'C',I2,'=',F10.6)
C
C     SET   THE MEMBER CODE TO IDENTIFY WHETHER THE MEMBER IS A BEAM
C     OR A COLUMN
C
      DO 75 MP=1,NOMP
      IPMM(MP)=1
      IF (WTFF(MP)) 74,75,75
   74 WTFF(MP)=-WTFF(MP)
      IPMM(MP)=-1
   75 CONTINUE
      WRITE(NWRIT,76) FLENG,T,WT,WD,WL
   76 FORMAT(1H1//10X,'SPAN LENGTH IS' ,F7.2,'FT'
     1//10X,'THICKNESS OF CONCRETE SLAB IS',F5.1,'INCH'
     2//10X,'TOTAL WORKING LOAD WT IS',F6.2,'K/FT',5X,'(DL=',F5.2,
     3'K/FT     LL=',F6.2,'K/FT)',
     4////10X,'COEFF. FOR OBJECTIVE FUN.      LOWER LIMITS'//)
      DO 77 MP=1,NOMP
   77 WRITE(NWRIT,78) MP,WTFF(MP),PMMIN(MP)
   78 FORMAT(/10X,'FOR MP',I2,F17.3,F17.3)
      IPPRT=IPRT
C
C     ASSUME THE SAFETY FACTOR SF AS FOLLOWS
C
      SF=1.7
      NELM=NTELM
      PU=SF*WT*FLENG
C
```

```
      LOOP=0
      DO 9000 IST=1,2
 1111 LOOP=LOOP+1
      IF (LOOP-1) 79,79,80
   79 IPRT=IPPRT
      GO TO 81
   80 IPRT=0
   81 IF (LOOP-5) 84,84,82
   82 WRITE(NWRIT,83)
   83 FORMAT(//10X,'CALCULATION IS STOPPED AFTER FIVE CIRCLES OF TRIAL')
      GO TO 9000
C
C     GENERATE THE ELEMENTARY HINGE ROTATION MATRIX
C
   84 DO 96 I=1,NELM
      IF (I-NBELM) 85,85,86
   85 TM=WEXTO(I)*PU*FLENG
      GO TO 87
   86 TM=WEXTO(I)*SF
   87 WEXT(I)=TM
      DO 96 J=1,NTHL
      ALP(J)=1.
      IF (MPT1(J)) 88,89,89
   88 GO TO (89,90),IST
   89 THETAE(I,J)=THETAO(I,J)
      GO TO 96
C
C     ASSUME THE RATIO MU/MP AS A FUNCTION OF INITIAL VALUE 'MP'
C
   90 K=MPT(J)
      X=PM(K,2)
      IF (ABS(X)-0.00001) 91,91,92
   91 ALP(J)=CO(1,1)+CO(2,1)*X+CO(3,1)*X**2+CO(4,1)*X**3
      ALP(J)=EXP(ALP(J)).
      GO TO 93
   92 X=ALOG(X)
      ALP(J)=CO(1,2)+CO(2,2)*X+CO(3,2)*X**2+CO(4,2)*X**3
   93 THETAE(I,J)=ALP(J)*THETAO(I,J)
   96 CONTINUE
      WRITE(NWRIT,119)
      GO TO (102,97),IST
```

```
   97 WRITE(NWRIT,98) L
   98 FORMAT(///10X,'COMPOSITE STEEL FRAMES'//
     110X,'CYCLE NO.',I3)
      WRITE(NWRIT,99)
   99 FORMAT(//10X,'ASSUMED RATIO OF MU/MP'/)
      DO 100 J=1,NTHL
  100 WRITE(NWRIT,101) J,ALP(J)
  101 FORMAT(10X,'MU/MP(',I3,')=',F7.3)
      GO TO 104
  102 WRITE(NWRIT,103)
  103 FORMAT(///10X,'NONCOMPOSITE STEEL FRAMES')
  104 WRITE(NWRIT,105)        NELM,NTHL,(MPT(J),J=1,NTHL)
  105 FORMAT (/'  ELEMENTARY HINGE ROTATION MATRIX IS '
     1          ,I4,' BY',I4//14(3X,'MP',I2,2X))
      DO 106 I=1,NELM
  106 WRITE (NWRIT,68) I,I,(THETAE(I,J),J=1,NTHL),WEXT(I)
C
C     GENERATE THE INITIAL INEQUALITIES
C
      DO 108 MP=1,NOMP
      DO 108 I=1,NELM
      WINT(I,MP)=0.
      DO 108 J=1,NTHL
      IF (MPT(J)-MP) 108,107,108
  107 WINT(I,MP)=WINT(I,MP)+ABS(THETAE(I,J))
  108 CONTINUE
C
C     SET THE INITIAL HINGE ROTATION MATRIX AND MECHANISM CODE DATA
C
      DO 109 I=1,NELM
      MECE(I)=MECO(I)
      WX(I,1)=WEXT(I)
      MEC(I,1)=MECE(I)
      DO 109 J=1,NTHL
  109 THETA(I,J,1)=THETAE(I,J)
C
C     DETERMINE THE ERROR CONTROL DATA 'EPS' FOR THE ELIMINATION
C     PROCEDURE
C
      EPS=999999.
      DO 115 I=1,NELM
      IF (WEXT(I)) 115,115,110
C
```

```
C     PICK UP THE [illegible]
C
  110 WIMIN=999999.
      DO 113 MP=1,NOMP
      IF (WINT(I,MP)) 113,113,111
  111 IF (WINT(I,MP)-WIMIN) 112,113,113
  112 WIMIN=WINT(I,MP)
  113 CONTINUE
      EPSI=WIMIN/WEXT(I)
      IF (EPSI-EPS) 114,115,115
  114 EPS=EPSI
  115 CONTINUE
      EPS=EPS*0.0001
      WRITE(NWRIT,116) NELM,NOMP
  116 FORMAT (1H1//'  MATRIX OF COLLAPSE MECHANISM INEQUALITIES',I4,' BY
     1 ',I4//)
      DO 117 I=1,NELM
  117 WRITE(NWRIT,118) I,(WINT(I,MP),MP=1,NOMP),WEXT(I)
  118 FORMAT(/'  NO.',I4/12(F11.4))
  119 FORMAT (1H1)
C
C     SET THE INITIAL VALUES FOR THE MECHANISM SORTING MATRIX AND E.T.C.
C
  121 DO 123 I=1,NELM
      DO 122 L=1,NELM
  122 MELM(I,L,1)=0
  123 MELM(I,L,1)=1
      IF (MINCM-1) 124,125,125
  124 KKKK=1
  125 MINCM1=MINCM+1
      IF (MXNHL) 126,126,127
  126 MXNHL=NTHL
C
C     GENERATE THE MATRIX OF HINGE ROTATIONS FOR COMBINED MECHANISMS
C
  127 ITOP=1
      IBOT=NELM
      K2=NELM
      K2P=NELM
      II2P=NELM
      NCNT=1
 2222 KK=IBOT
      II2=0
```

```
      DO 275 J=1,NTHL
      DO 275 I=1,NELM
      II1=0
      IF (THETAE(I,J)) 128,275,128
  128 DO 270 IK=ITOP,IBOT
      II1=II1+1
      IF (MECE(I)*MEC(II1,1)) 270,129,129
  129 IF (MELM(II1,I,1)) 130,130,270
  130 RTHTA=THETA(II1,J,1)/THETAE(I,J)
      IF (RTHTA) 131,270,270
C
C     COMBINE TWO MECHANISMS WITH OPPOSITE HINGE ROTATION EACH OTHER
C
  131 RTHTA=ABS(RTHTA)
C
C     GENERATE THE NEW MECHANISM BY ADDING TWO EQUATIONS AFTER MODIFING
C     THE EQUATION WITH LARGER COEFFICIENTS
C
      IT=I
      JT=IK
      IF (RTHTA-1.0) 145,145,132
  132 WEXT1=WEXT(I)+WX(II1,1)/RTHTA
      TI=1.
      TJ=1./RTHTA
C
C     ELIMINATE THIS COMBINED MECHANISM IF THE CORRESPONDING EXTERNAL
C     WORK OR TOTAL INTERNAL WORK IS ZERO.
C
      IF (WEXT1-EPS) 270,270,133
  133 DO 134 JJ=1,NTHL
      THETA1(JJ)=THETA(II1,JJ,1)/RTHTA
  134 CONTINUE
      KK=KK+1
      K2=K2+1
      II2=II2+1
      IF (K2-MXELM) 135,4444,4444
  135 SUM=0.
      DO 140 JJ=1,NTHL
      THETA(II2,JJ,2)=THETAE(I,JJ)+THETA1(JJ)
      SUM=SUM+DABS(THETA(II2,JJ,2))
  140 CONTINUE
      GO TO 161
  145 WEXT1=WEXT(I)*RTHTA+WX(II1,1)
```

```
      II=NTHTA
      TJ=1.
C
C     ELIMINATE THE COMBINED MECHANISM IF THE CORRESPONDING EXTERNAL
C     WORK CR TOTAL INTERNAL WORK IS ZERO.
C
      IF (WEXT1-EPS) 27C,270,147
  147 DO 150  JJ=1,NTHL
      THETA1(JJ)=THETAE(I,JJ)*RTHTA
  150 CONTINUE
      KK=KK+1
      K2=K2+1
      II2=II2+1
      IF (K2-MXELM) 155,4444,4444
  155 SUM=0.
      DO 160  JJ=1,NTHL
      THETA(II2,JJ,2)=THETA1(JJ)+THETA(II1,JJ,1)
      SUM=SUM+DABS(THETA(II2,JJ,2))
  160 CONTINUE
  161 KK1=KK-1
      II21=II2-1
      K21=K2-1
      WX(II2,2)=WEXTI
      K=MECE(I)+MEC(II1,1)
      IF (K) 162,163,164
  162 MEC(II2,2)=-1
      GO TO 165
  163 MEC(II2,2)=0
      GO TO 165
  164 MEC(II2,2)=1
  165 IF (SUM-EPS) 255,255,166
C
C     ELIMINATE THE DUPLICATED COMBINED MECHANISM
C
  166 DO 170 K=1,II2P
      IF (WX(K,1)-EPS) 170,170,167
  167 DO 168  JJ=1,NTHL
      TM1=DABS(THETA(II2,JJ,2))/WX(II2,2)
      TM2=DABS(THETA(K,JJ,1))/WX(K,1)
      IF  (ABS(TM1-TM2)-EPS) 168,168,170
  168 CONTINUE
      GO TO 255
  170 CONTINUE
```

```
      IF (II-1) [illegible]
  172 DO 176 K=1,II2
      IF (WX(K,2)-EPS) 17?,17?,17?
  173 DO 174 JJ=1,NTHL
      TM1=DABS(THETA(II2,JJ,2))/WX(II2,2)
      TM2=DABS(THETA(K,JJ,2))/WX(K,2)
      IF  (ABS(TM1-TM2)-EPS) 174,174,176
  174 CONTINUE
      GO TO 255
  176 CONTINUE
C
C     ELIMINATION OF MECHANISMS WITH LESS HINGES THAN THE MINIMUM
C     REQUIRED HINGES OR WITH MORE HINGES THAN THE MAXIMUM POSSIBLE HING
C
  178 NTHG=0
      DO 196 JJ=1,NTHL
      IF (THETA(II2,JJ,2)) 195,196,195
  195 NTHG=NTHG+1
  196 CONTINUE
      IF (NTHG-MXNHL) 198,198,255
  198 IF (NTHG-MINHL) 255,201,201
C
C     ELIMINATE THE COLLAPSE MECHANISM WITH STATICALLY UNSTABLE JOINT
C     ...CHECK BY JOINT LATAS
  201 DO 209 JJ=1,NTHL
C
      IF (JNT(JJ)-JJ) 209,202,209
  202 JNTL=1
      IF (DABS(THETA(II2,JJ,2))-EPS) 209,209,203
  203 JNTL1=1
      JJ1=JJ+1
      IF (NTHL-JJ1) 209,204,204
  204 DO 208 JJJ=JJ1,NTHL
      IF (JNT(JJ)-JNT(JJJ)) 208,205,208
  205 JNTL=JNTL+1
      IF (DABS(THETA(II2,JJJ,2))-EPS) 209,209,206
  206 JNTL1=JNTL1+1
  208 CONTINUE
      IF (JNTL-JNTL1) 255,255,209
  209 CONTINUE
C
C     GENERATE THE COMBINED COLLAPSE INEQUALITIES IN TERMS OF
C     PLASTIC MOMENTS
```

```
      DO 215 MP=1,NOMP
      WINT(K2,MP)=0.
      DO 215 JJ=1,NTHL
      IF (MPT(JJ)-MP) 215,210,215
  210 WINT(K2,MP)=WINT(K2,MP)+DABS(THETA(II2,JJ,2))
  215 CONTINUE
      WEXT(K2)=WX(II2,2)
      NO(II2)=K2
C
C     ELIMINATE THE NEWLY COMBINED MECHANISM PRESENTING BIGGER VALUES OF
C     NORMALIZED INTERNAL WORK THAN THE PREVIOUS ONES
C     BY FORWARD ELIMINATION
C
      DO 230 K=1,K2P
      SUM=0.
      IF (WEXT(K)-EPS) 230,230,220
  220 DO 227 MP=1,NOMP
      IF (WINT(K2,MP)-EPS) 224,221,221
  221 DNRWI=WINT(K2,MP)/WEXT(K2)-WINT(K,MP)/WEXT(K)
      IF (ABS(DNRWI)-EPS) 227,222,222
  222 IF (DNRWI) 230,227,223
  223 SUM=SUM+DNRWI
      GO TO 227
  224 IF (WINT(K,MP)-EPS) 227,227,230
  227 CONTINUE
      IF (SUM) 228,228,255
  228 DO 229 MP=1,NOMP
  229 WINT(K2,MP)=0.
      WEXT(K2)=0.
      K2=K21
      NO(II2)=9999999
      GO TO 236
  230 CONTINUE
  236 K2P=K2
      IF (II2-MXMELM) 238,238,4444
  238 DO 240 L=1,NELM
  240 MELM(II2,L,2)=MELM(II1,L,1)
      MELM(II2,I,2)=1
      IF (IPRT) 270,270,245
  245 WRITE (NWRIT,246) KK,TI,IT,TJ,JT,MEC(II2,2)
  246 FORMAT(/'  COLLAPSE MECHANISM NO.',I4,'  IS  COMBINED  (',
     1 F8.3,'* NO.',I4,') WITH  (',F8.3,'* NO.',I4,')      MECHANISM CO
```

```
      [illegible]
      [illegible]
  255 WX(II2,2)=[illegible]
      DO 260 JJ=1,NTHL
  260 THETA(II2,JJ,2)=0.
      DO 265 MP=1,NOMP
  265 WINT(K2,MP)=0.
      WEXT(K2)=0.
      KK=KK1
      II2=II21
      K2=K21
  270 CONTINUE
  275 CONTINUE
      II2P=II2
      KKK=KK
      IF (KKK.EQ.IBOT) GO TO 3333
      IBOT1=IBOT+1
      ITOP=IBOT1
      IBOT=KKK
      ICOMB=IBOT-ITOP+1
      II2=0
      DO 295 I=ITOP,IBOT
      II2=II2+1
      WX(II2,1)=WX(II2,2)
      MEC(II2,1)=MEC(II2,2)
      DO 293 J=1,NTHL
  293 THETA(II2,J,1)=THETA(II2,J,2)
      DO 294 L=1,NELM
  294 MELM(II2,L,1)=MELM(II2,L,2)
  295 CONTINUE
      NCNT=NCNT+1
      IF (NCNT-MINCM) 300,297,[illegible]
  297 KKKK=ITOP
  300 IF (NCNT-MXNCM) 2222,[illegible],[illegible]
 3333 CONTINUE
      WRITE (NWRIT,[illegible])
      WRITE(NWRIT,305)
  305 FORMAT (//'  ALL POSSIBLE COMBINATIONS HAVE [illegible])
      GO TO 320
 4444 WRITE (NWRIT,[illegible])
      WRITE (NWRIT,315)
  315 FORMAT(/'  STOP GENERATING THE MATRIX [illegible] FOR COMBI
     1NED MECHANISM SO AS NOT TO EXCEED LIMITED SIZE OF DIMENSION')
```

```
      IJ=NJ
C
C     INTRODUCE THE [illegible]
C
      DO 350 MP=1,NOMP
      IF (PMMIN(MP)) 350,350,340
  340 IJ=IJ+1
      DO 345 MP1=1,NOMP
  345 WINT(IJ,MP1)=0.
      WINT(IJ,MP)=1.
      WEXT(IJ)=PMMIN(MP)
  350 CONTINUE
      WRITE(NWRIT,365)IJ
  365 FORMAT('  THE NUMBER OF POSSIBLE MECHANISMS =',I4)
C
C     NORMALIZE THE EXTERNAL WORK TO UNIT 1
C
      DO 390 I=1,IJ
      IF (WEXT(I)-EPS) 390,390,387
  387 DO 388 MP=1,NOMP
      WINT(I,MP)=WINT(I,MP)/WEXT(I)
  388 CONTINUE
      WEXT(I)=1.
  390 CONTINUE
      IF (IPRT) 400,400,391
  391 WRITE(NWRIT,116) IJ,NOMP
      DO 392 I=1,IJ
  392 WRITE(NWRIT,118) I,(WINT(I,MP),MP=1,NOMP),WEXT(I)
      WRITE(NWRIT,394)
  394 FORMAT(//10X,'THESE ARE POSSIBLE COLLAPSE INEQUALITIES NORMALIZED
     1 AFTER BACKWARD ELIMINATION'/
     210X,'TO BE USED FOR GENERATING FEASIBLE COLLAPSE INEQUALITIES')
C
C     GENERATE THE MATRIX OF FINAL COLLAPSE INEQUALITIES
C     FOR BOTH ORIGINAL AND PARTICULAR SOLUTIONS
C     AND PROCEED TO THE FINAL CALCULATIONS
C
  400 NCOMP=NOMP
      DO 8888 LLL=1,NP
      DO 405 J=1,NOMP
      WTF(J)=WTFF(J)
      IPM(J)=IPMM(J)
```

```
      [illegible]
      WNT(I,J)=WINT(I,J)
  400 CONTINUE
      DO 410 I=1,IJ
  410 WXT(I)=WEXT(I)
      IF (NOMP.EQ.1) GO TO 464
      IF (LLL.EQ.1) GO TO 470
C
C     PARTICULAR SOLUTIONS
C
      NOMP1=NOOMP
      READ (NREAD,10) (NMP(J),J=1,NOOMP)
      WRITE(NWRIT,415) LLL,(J,NMP(J),J=1,NOOMP)
  415 FORMAT('  PARTICULAR SOLUSION ',I3//9('   MP',I2,'=MP',I3,2X))
      DO 425 J=1,NOOMP
  425 NREC(J)=0
      DO 440 J=1,NOOMP
      J1=J+1
      IF  (J1.GT.NOOMP) GO TO 440
      DO 440 JJ=J1,NOOMP
      IF(NMP(J)-NMP(JJ)) 440,430,440
  430 NOMP1=NOMP1-1
      WTF(J)=WTF(J)+WTF(JJ)
      WTF(JJ)=0.
      DO 435 I=1,IJ
      WNT(I,J)=WNT(I,J)+WNT(I,JJ)
      WNT(I,JJ)=0.
  435 CONTINUE
      NREC(JJ)=JJ
  440 CONTINUE
      IF (IPRT) 448,448,442
  442 WRITE(NWRIT,116) IJ,NOOMP
      DO 445 I=1,IJ
  445 WRITE(NWRIT,118) I,(WNT(I,J),J=1,NOOMP),WXT(I)
      WRITE(NWRIT,447) LLL
  447 FORMAT(//10X,'THESE ARE NORMALIZED POSSIBLE INEQUALITIES BEFORE EL
     1IMINATING  DUPLICATED PLASTIC MOMENT  TERMS FOR SOLUTION NO.',I3)
  448 JJ=0
      DO 457 J=1,NOOMP
      IF (NREC(J)) 457,450,457
  450 JJ=JJ+1
      WTF(JJ)=WTF(J)
      IPM(JJ)=IPM(J)
```

```
  458 WXT(I,JJ)=WXT(I,J)
  457 CONTINUE
      NOMP=JJ
      IF (IPRT) 463,...
  458 WRITE (NWRIT,116) IJ,NOMP
      DO 460 I=1,IJ
  460 WRITE (NWRIT,118) I,(WNT(I,J),J=1,NOMP),WXT(I)
      WRITE(NWRIT,461) LLL
  461 FORMAT(//10X,'THESE ARE NORMALIZED POSSIBLE INEQUALITIES  AFTER EL
     1IMINATING  DUPLICATED PLASTIC MOMENT  TERMS FOR SOLUTION NO.',I3)
  463 IF (NOMP-1) 464,464,470
  464 PM(1,IST)=WXT(1)/WNT(1,1)
      DO 466 I=2,IJ
      TM=WXT(I)/WNT(I,1)
      IF (PM(1,IST)-TM) 465,465,466
  465 PM(1,IST)=TM
  466 CONTINUE
      OBFN(IST)=WTF(1)*PM(1,IST)
      WRITE(NWRIT,467) OBFN(IST)
  467 FORMAT(1H1//' OBJ. FUNCTION',F20.5//)
      WRITE(NWRIT,468)
  468 FORMAT(' VARIABLE              VALUE'/)
      WRITE(NWRIT,469) NOMP,PM(1,IST)
  469 FORMAT(I5,F25.5)
      FL(1)=WTF(1)
      GO TO 8888
C
C     ELIMINATE IDENTICAL EQUATIONS
C
  470 IJ1=IJ
      DO 500 I=1,IJ
      IF (WXT(I)-EPS) 500,500,471
  471 I1=I+1
      IF(I1.GT.IJ)GO TO 500
      DO 495 II=I1,IJ
      IF (WXT(II)-EPS) 495,495,472
C
C     ELIMINATE THE COMBINED INEQUALITIES WITH BIGGER VALUES OF
C     NORMALIZED INTERNAL WORK THAN THE OTHERS.
C     BY FORWARD ELIMINATIONS
C
  472 DO 476 J=1,NOMP
```

```
      IF  (WNT(I,J)- [illegible]
  473 DNRWI=WNT(I,J)-WNT(II,J)
      IF  (DNRWI+EPS) 4[illegible],474,474
  474 IF  (WNT(II,J)-EPS) 476,476,480
  476 CONTINUE
      WXT(I)=0.
      DO 478 J=1,NOMP
  478 WNT(I,J)=0.
      GO TO 500
C
C     BACKWARD ELIMINATION
C
  480 DO 484 J=1,NOMP
      IF  (WNT(II,J)-EPS) 483,482,482
  482 DNRWI=WNT(II,J)-WNT(I,J)
      IF  (DNRWI+EPS) 495,484,484
  483 IF  (WNT(I,J)-EPS) 484,484,495
  484 CONTINUE
      WXT(II)=0.
      DO 490 J=1,NOMP
  490 WNT(II,J)=0.
  495 CONTINUE
  500 CONTINUE
      IF  (IPRT) 508,508,502
  502 WRITE(NWRIT,116) IJ,NOMP
      DO 505 I=1,IJ
  505 WRITE (NWRIT,118) I,(WNT(I,J),J=1,NOMP),WEXT(I)
      WRITE(NWRIT,507) LLL
  507 FORMAT(//10X,'THESE ARE NORMALIZED FEASIBLE COLLAPSE INEQUALITIES
     1 AFTER OVERALL ELIMINATION FOR SOLUTION NO.',I3)
C
C     GENERATE THE FEASIBLE COLLAPSE INEQUALITIES
C     ELIMINATION FOR THE JOINT ELEMENTARY MECHANISMS ARE EXECUTED ,TOO.
C
  508 II=0
      DO 525 I=1,IJ
      IF  (WXT(I)) 510,525,510
  510 II=II+1
      WXT(II)=WXT(I)
      DO 515 J=1,NOMP
  515 WNT(II,J)=WNT(I,J)
  525 CONTINUE
      IJ1=II
```

```
      IF (IM   )
  5?? WRITE(NWRIT,5??) I
      DO 535 I=1,IJ1
  535 WRITE (NWRIT,5?8) I,(WNT(I,J),J=1,NOMP),AXT(I)
      WRITE(NWRIT,540) LBL
  540 FORMAT(//10X,'THESE ARE NORMALIZED FEASIBLE COLLAPSE INEQUALITIES
     1CONDENSED AFTER OVERALL ELIMINATION FOR SOLUTION NO.',I3/10X,
     2 'TO BE USED IN LINEAR PROGRAMMING')
C
C     STORE THE LENGTH OF EACH MEMBER FOR CALCULATING THE TOTAL WEIGHT
C
  545 DO 550 J=1,NOMP
  550 FL(J)=WTF(J)
C
C     CHECK IF THERE IS AN EQUATION PARALLEL TO THE OBJECTIVE FUNCTION
C
      NOMP1=NOMP-1
      DO 555 I=1,IJ1
      DO 552 J=1,NOMP1
      J1=J+1
      IF (WNT(I,J1)-EPS) 555,555,551
  551 TM1=WNT(I,J)/WNT(I,J1)
      TM2=WTF(J)/WTF(J1)
      IF (ABS(TM1-TM2)-0.0001*TM1) 552,552,553
  552 CONTINUE
      WRITE(NWRIT,553) I
  553 FORMAT(//10X,'IT IS NECESSARY TO INVESTIGATE THE POSSIBILITY OF IN
     1DEFINITE SOLUTIONS,'/10X,'SINCE EQUATION NO.',I4,' IS PARALLEL TO
     2THE OBJECTIVE FUNCTION.'/)
      JJN=NOMP+1
      GO TO 557
  555 CONTINUE
  556 JJN=1
C
C     IF THERE IS AN EQUATION PARALLEL TO OBJECTIVE FUNCTION,
C     IT IS POSSIBLE FOR THE SOLUTION TO BE INDEFINITE.
C     HENCE INVESTIGATE SEVERAL SOLUTIONS AT BOUNDARY POINTS
C     BY MULTIPLING EACH COEFFICIENTS OF THE OBJECTIVE FUNCTION
C     BY 1.0001
C
  557 DO 7777 JJJ=1,JJN
C
C     GENERATE THE DATA  FOR SUBROUTINE 'LINEAR'
```

```
      I. (N[illegible]-1) [illegible]
  558 L=0
      DO 560 I=1,IJ1
      L=L+1
      A(L)=-WXT(I)
      DO 559 J=1,NCMP
      L=L+1
      A(L)=WNT(I,J)
  559 CONTINUE
      L=L+1
      A(L)=0.
  560 CONTINUE
      DO 570 J=1,NCMP
      L=L+1
      A(L)=0.
      LJ=L+J
      DO 565 JJ=1,NCMP
      L=L+1
  565 A(L)=0.
      A(LJ)=1.
      L=L+1
  570 A(L)=0.
      L=L+1
      A(L)=0.
      DO 585 J=1,NCMP
      L=L+1
      IF (JJJ-1) 580,580,575
  575 IF (J-JJJ+1) 580,578,580
  578 A(L)=WTF(J)*1.0001
      GO TO 585
  580 A(L)=WTF(J)
  585 CONTINUE
      L=L+1
      A(L)=0.
      III=NUMP+2
      JJ=IJ1+NCMP+1
      CALL LINEAR (A,III,JJ,JJJ)
  587 IF (LLL-1) 590,590,7777
  590 IF (JJJ-1) 595,595,7777
  595 I=III*(JJ-1)+1
      OBFN(IST)=A(I)
      LOOP1=0
```

```
      [illegible]
      K=II*(JJ-NOMP+J-2)+
      GO TO (630,610),IST
  610 IF (A(K)-0.00001) 612,615,615
  612 A(K)=0.00001
  615 TM=A(K)-PM(J,IST)
      TM=TM/A(K)
      IF (ABS(TM)-0.05) 630,620,620
  620 LOOP1=1
  630 PM(J,IST)=A(K)
  640 CONTINUE
 7777 CONTINUE
 8888 NOMP=NOOMP
      GO TO (650,670),IST
  650 DO 660 J=1,NOMP
  660 PM(J,2)=PM(J,1)
      GO TO 9000
  670 IF (LOOP1) 9000,9000,1111
 9000 CONTINUE
      WRITE(NWRIT,820)
  820 FORMAT(1H1//16X,'MP(NON-COMP.)',16X,'MP(COMP.)'/
     113X,'(K.FT)    (WT2.L)',9X,'(K.FT)    (WT2.L)'//)
      DO 850 I=1,NOMP
      IF (PM(I,1)-0.00001) 825,827,827
  825 PM(I,1)=0.00001
  827 TM=PM(I,2)/PM(I,1)
      WTL2=(WT*SF)*FLENG**2
      TM1=PM(I,1)/WTL2
      TM2=PM(I,2)/WTL2
      IF (IPMM(I)) 840,830,830
  830 WRITE(NWRIT,835) I,PM(I,1),TM1,PM(I,2),TM2,TM
  835 FORMAT(I4,2(F15.3,F10.5),F15.5,'...BEAM MEMBER')
      GO TO 850
  840 WRITE(NWRIT,845) I,PM(I,1),TM1,PM(I,2),TM2,TM
  845 FORMAT(I4,2(F15.3,F10.5),F15.5,'...COLUMN MEMBER')
  850 CONTINUE
 9999 STOP
      END
```

```
0001          SUBROUTINE LINEAR (A,III,JJ,JJJ)
        C     SIMPLEX METHOD FOR SOLVING LINEAR PROGRAMMING
        C     III=TOTAL NUMBER OF ROWS OF THE SIMPLEX TABLEAU
        C     II=III-1
        C     JJ=TOTAL NUMBER OF COLUMNS OF THE GIVEN ARGUMENTED TABLEAU
        C     JJJ=CONTROL DATA FOR THE INDEFINITE PROBLEM
        C
        C     NEXT STATEMENT FOR INITIALIZATION
        C
0002          DOUBLE PRECISION A(III,JJ),W(50),X,XMIN,OBF(50)
0003          DIMENSION L(50)
0004          NREAD=1
0005          NWRIT=3
0006          II=III-1
0007          DO 3 I=2,II
0008          K=I-1
0009        3 OBF(K)=A(I,JJ)
0010        5 FORMAT(10X,9F14.5)
0011          DO 10 I=1,III
0012          W(I)=0.
0013       10 L(I)=0
0014          DO 15 I=2,II
0015       15 L(I)=JJ-III+I
0016          I=II-1
0017          IF (JJJ-1) 18,18,3
0018       18 WRITE(NWRIT,20) I,III,JJ
0019       20 FORMAT(1H1//1X,'NUMBER OF CONSTRAINS IS ',I3//1X,
          1'NUMBER OF ROWS IN MATRIX A(III,JJ) IS ',I3//1X,
          2'NUMBER OF COLUMNS IN MATRIX A(III,JJ) IS ',I3//)
0020       31 CONTINUE
0021       32 FORMAT(1H1)
0022          KKK=0
        C
        C     THE NEXT STATEMENTS ARE TO LOOK FOR THE ROW AT WHICH THERE IS
        C     NO SLACK VARIABLE (NOT INCLUDING THE FIRST ROW).
        C
0023       35 I=1
0024       40 I=I+1
0025          IF (I-III) 45,65,65
0026       45 IF (L(I)) 40,50,40
        C
        C     CALCULATE
        C     NEW LAST ROW=LAST ROW- THE ROW WITHOUT SLACK VARIABLE
```

```
0027         50 DO 60 J=1,JJ
0028            IF (A(I,J)) 55,60,55
0029         55 A(III,J)=A(III,J)-A(I,J)
0030         60 CONTINUE
0031            GO TO 40
          C
          C     NEXT STATEMENTS FOR SEARCHING FOR THE COLUMN AT WHICH THE MOST
          C     NEGATIVE ENTRY APPEARS EITHER IN THE FIRST(OBJECTIVE FUNCTION)
          C     OR LAST(FORM P) ROW.
          C
0032         65 K=III
0033         70 J=0
0034            W(K)=0.
0035            L(K)=0
0036         75 J=J+1
0037            IF (J-JJ) 80,95,95
0038         80 IF (A(K,J)) 85,75,75
0039         85 IF (W(K)-A(K,J)) 75,75,90
0040         90 W(K)=A(K,J)
0041            L(K)=J
0042            GO TO 75
          C
          C     TEST FOR L(K). IF L(K) IS EQUAL TO.ZERO , THAT IS , ALL THE
          C     ENTRIES EXCEPT THE EXTREME RIGHT ONE EITHER IN THE FIRST OR LAST
          C     ROW ARE POSITIVE, GO TO ST.62 FOR FURTHER EXAMINATION.
          C
0043         95 IF (L(K)) 100,195,100
          C
          C     FIND OUT THE PIVOT COLUMN
          C
0044        100 KJ=L(K)
          C
          C     TEST EVERY ENTRY IN THE PIVOT COLUMN TO SEE IF IT IS POSITIVE
          C     OR NOT. IF IT IS, GO TO ST.121 TO COMPUTE THE RATIO DEFINED IN
          C     THE LAST SECTION
          C
0045            DO 105 I=2,II
0046            IF (A(I,KJ)) 105,105,115
0047        105 CONTINUE
          C
          C     IF ALL THE ENTRIES IN THE PIVOT COLUMN ARE ZERO OR NEGATIVE
          C     NUMBERS, 'UNBOUNDED' IS GOING TO BE TYPED.
```

```
0048          WRITE(NWRIT,110)
0049      110 FORMAT(' FEASIBLE')
0050          GO TO 240
        C
        C     THE FOLLOWING STATEMENTS ARE FOR COMPUTING THE RATIO DEFINED
        C     AND FOR DETERMINING THE LOCATION OF THE PIVOT.
        C
0051      115 I=1
0052          JK=0
0053      120 I=I+1
0054          IF (I-II) 125,125,145
0055      125 IF (A(I,KJ)) 120,120,130
0056      130 X=A(I,JJ)/A(I,KJ)
0057          IF (JK) 135,140,135
0058      135 IF (X-XMIN) 140,120,120
0059      140 XMIN=X
0060          JK=I
0061          GO TO 120
        C
        C     THE NEXT STATEMENT INDICATES THE PIVOT ELEMENT BEFORE NORMALIZATION
        C
0062      145 X=A(JK,KJ)
0063          L(JK)=KJ
        C
        C     THE NEXT STATEMENT FOR CALCULATING THE NEW ROWS ABOVE THE PIVOT ROW
        C
0064          DO 150 I=1,III
0065      150 W(I)=A(I,KJ)
0066          IJ=JK-1
0067          DO 165 I=1,IJ
0068          DO 165 J=1,JJ
0069          IF (A(JK,J)) 155,165,155
0070      155 IF (W(I)) 160,165,160
0071      160 A(I,J)=A(I,J)-W(I)*(A(JK,J)/X)
0072      165 CONTINUE
        C
        C     NEXT STATEMENTS FOR CALCULATING THE NEW ROWS BELOW THE PIVOT ROW
        C
0073          IJ=JK+1
0074          DO 180 I=IJ,III
0075          DO 180 J=1,JJ
0076          IF (A(JK,J)) 170,180,170
```

```
  175 IF (A(I)) 175,175,...
  175 A(I,J)=A(I,J)-A(I)*A(JK,J)/X
  180 CONTINUE
C
C     NEXT STATEMENTS FOR NORMALIZATION
C
      DO 185 J=1,JJ
  185 A(JK,J)=A(JK,J)/X
      KKK=KKK+1
      WRITE(NWRIT,190) KKK,A(K,JJ),L(JK)
  190 FORMAT(2X,I4,6X,F15.2,10X,I4)
      GO TO 70
C
C     NEXT STATEMENT FOR TESTING TO SEE IF IT IS THE FIRST ROW ON
C     WHICH ALL THE ENTRIES ARE POSITIVE EXCEPT THE EXTREME RIGHT ONE
C     IF IT IS, THAT MEANS, NO FURTHER IMPROVEMENT ON THE SOLUTION
C     CAN BE MADE, GO TO ST.70 AND THE ANSWER WILL BE TYPED OUT.
C
  195 IF (K-1) 240,240,200
  200 IJ=JJ-1
C
C     TEST TO SEE WHETHER ALL THE ELEMENTS ON THE LAST ROW(NOT
C     INCLUDING THE EXTREME RIGHT ONE) ARE CLOSE TO ZERO.
C     IT IS DEFINED IN THE NEXT STATEMENTS THAT THE PROBLEM IS
C     INFEASIBLE IF ONE(OR MORE) OF THEM IS LARGER THAN 0.0001.
C
      DO 205 J=1,IJ
      IF (A(K,J)-0.00001) 205,205,225
  205 CONTINUE
      WRITE(NWRIT,210)
  210 FORMAT(//' UNBOUNDED')
      WRITE(NWRIT,215)
  215 FORMAT(' ITERATION     OBJ. FUNCTION      NEW BASIC VAR.')
C
C     IF,AFTER ITERATIONS,ALL THE ELEMENTS IN THE LAST ROW HAVE
C     BECOME POSITIVE BUT NEAR ZERO. DEFINE ALL OF THEM TO BE ZERO.
C
      DO 220 J=1,JJ
  220 A(III,J)=0.
C
C     IN CASE OF NONARTIFICIAL PROBLEM,DEFINE K=1,AND GO TO ST.44
C     FOR SEARCHING FOR THE PIVOT COLUMN.
C
```

```
      K=0
      KKK=0
      GO TO 70
C
C     TYPE OUT THE SOLUTION
C
  225 CONTINUE
      WRITE(NWRIT,230)
  230 FORMAT(//' INFEASIBLE')
  240 IF (JJJ-1) 242,242,247
  242 WRITE(NWRIT,245) A(1,JJ)
  245 FORMAT(///' OBJ. FUNCTION',F20.5/)
      GO TO 253
  247 WRITE(NWRIT,248)
  248 FORMAT(//' GOVERING INEQUALITIES'/)
      K=JJ-III+1
      DO 251 J=1,K
      IF (A(1,J)) 251,249,251
  249 WRITE(NWRIT,250) J
  250 FORMAT(2X,'EQ. NO.',I4)
  251 CONTINUE
      K=JJJ-1
      WRITE(NWRIT,252) K,A(1,JJ)
  252 FORMAT(///' PART. OBJ. FUNCTION (NO.',I3,')',F20.5/)
  253 WRITE(NWRIT,254)
  254 FORMAT(' VARIABLE                VALUE                INPUT COST')
      NOMP=III-2
      DO 255 I=1,NOMP
      K=JJ-NOMP+I-1
  255 WRITE(NWRIT,260) I,A(1,K),OBF(I)
  260 FORMAT(I4,2F20.5)
 9999 RETURN
      END
```

F88-LEVEL LINKAGE EDITOR OPTIONS SPECIFIED MAP,LIST,LET
DEFAULT OPTION(S) USED - SIZE=(102400,12288)

MODULE MAP

CONTROL SECTION NAME	ORIGIN	LENGTH	ENTRY NAME	LOCATION	NAME	LOCATION	NAME	LOCATION	NAME	LOCATION
MAIN*	00	62C22								
LINEAR	62C28	12FC								
IHCSLOG *	63F28	186	ALOG10	63F28	ALOG	63F40				
IHCSEXP *	640F0	192	EXP	640F0						
IHCFCOMH*	64278	F43	IBCOM#	64278	FDIOCS#	64334	INTSWTCH	[illegible]		
IHCCOMH2*	651C0	650	SEQDASD	65538						
IHCFCVTH*	65820	1190	ADCON#	65820	FCVAOUTP	659CA	FCVLOUTP	65D5A	FCVZOUTP	65AAA
			FCVIOUTP	65F58	FCVEOUTP	6635A	FCVCOUTP	66574	INT6SWCH	6595B
IHCFINTH*	669C0	512	ARITH#	669C0	ADJSWTCH	66D2C				
IHCEFIOS*	66ED8	1378	FIOCS#	66ED8	FIOCSBEP	66EDE				
IHCERRM *	68250	53C	ERRMON	68250	IHCERRE	68268				
IHCUOPT *	68810	3CC								
IHCETRCH*	68810	28E	IHCTRCH	68810	ERRTRA	68B18				
IHCUATBL*	68DA0	148								

ENTRY ADDRESS 00
TOTAL LENGTH 68EE8

****MAIN DOES NOT EXIST BUT HAS BEEN ADDED TO DATA SET

A. 4-4 Output Example for the Linear Optimization of the Composite Frame in Sec. 5-2

[illegible]

[illegible]

TOTAL WORKING [illegible] ([illegible])

COEFF. FOR OBJECTIVE FUN.		LOWER LIMITS
FOR MP 1	60.000	0.0
FOR MP 2	60.000	0.0
FOR MP 3	60.000	0.0
FOR MP 4	72.000	104.100
FOR MP 5	36.000	179.700

APPROXIMATED FUNCTION FOR MU/MP

CASE 1 : MU/MP=EXP(C1+C2*MP+C3*MP**2+C4*MP**3)

C 1= 0.835428
C 2= -0.000697
C 3= 0.000000
C 4= 0.0

CASE 2 : MU/MP=C1+C2*LOGMP+C3*(LOGMP)**2+C4*(LOGMP)**3

C 1= 7.532917
C 2= -1.947446
C 3= 0.223332
C 4= -0.009424

JOINT NO.	CODE
1	-1
2	2
3	2
4	4
5	4
6	6
7	-7
8	8
9	8
10	10
11	10
12	12
13	-13
14	14
15	14
16	16
17	16
18	18
19	5
20	-20
21	12
22	12
23	-23
24	18
25	4
26	-26
27	10
28	10
29	-29
30	10
31	2
32	-32
33	3
34	8
35	-35
36	14

WHERE THE NEGATIVE JOINT CODE NUMBER SHOWS A SINGLE JOINT.

```
THIS IS ELEMENTARY MECHANISM NO. =    1

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE =  0

THIS HINGE IS AT SECTION  19   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  20   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  21   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1

THIS IS ELEMENTARY MECHANISM NO. =    2

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE =  0

THIS HINGE IS AT SECTION   6   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  20   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  21   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1

THIS IS ELEMENTARY MECHANISM NO. =    3

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE =  0

THIS HINGE IS AT SECTION  22   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  23   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  24   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1

THIS IS ELEMENTARY MECHANISM NO. =    4

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE =  [illegible]

THIS HINGE IS AT SECTION  22   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP [illegible]
THIS HINGE IS AT SECTION  23   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP [illegible]
THIS HINGE IS AT SECTION  16   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
```

```
THIS IS ELEMENTARY MECHANISM NO. =    5

THIS MECHANISM HAS   3  HINGES    EXTERNAL WORK =     0.2500(SF*W*L2)   MECHANISM CODE =   0

THIS HINGE IS AT SECTION  25    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  26    ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  27    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2

THIS IS ELEMENTARY MECHANISM NO. =    6

THIS MECHANISM HAS   3  HINGES    EXTERNAL WORK =     0.2500(SF*W*L2)   MECHANISM CODE =   0

THIS HINGE IS AT SECTION  28    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  29    ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  30    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2

THIS IS ELEMENTARY MECHANISM NO. =    7

THIS MECHANISM HAS   3  HINGES    EXTERNAL WORK =     0.2500(SF*W*L2)   MECHANISM CODE =   0

THIS HINGE IS AT SECTION  31    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  32    ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  33    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3

THIS IS ELEMENTARY MECHANISM NO. =    8

THIS MECHANISM HAS   3  HINGES    EXTERNAL WORK =     0.2500(SF*W*L2)   MECHANISM CODE =   0

THIS HINGE IS AT SECTION  34    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  35    ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  36    ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
```

```
THIS IS ELEMENTARY MECHANISM NO. =   9

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION   4   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   5   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  26   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  27   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2

THIS IS ELEMENTARY MECHANISM NO. =  10

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION  28   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  29   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  16   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  17   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4

THIS IS ELEMENTARY MECHANISM NO. =  11

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION   2   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   3   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  32   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  33   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3

THIS IS ELEMENTARY MECHANISM NO. =  12

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.2500(SF*W*L2)  MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION  34   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  35   ANGLE ROTATION =   2.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  14   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  15   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
```

THIS IS ELEMENTARY MECHANISM NO. = 13

THIS MECHANISM HAS 6 HINGES EXTERNAL WORK = 54.0000(SF) MECHANISM CODE = 0

THIS HINGE IS AT SECTION 5 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 6 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 11 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION 12 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION 17 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 18 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4

THIS IS ELEMENTARY MECHANISM NO. = 14

THIS MECHANISM HAS 6 HINGES EXTERNAL WORK = 54.0000(SF) MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION 5 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 11 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION 12 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION 17 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 19 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP [illegible]
THIS HINGE IS AT SECTION 24 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP [illegible]

THIS IS ELEMENTARY MECHANISM NO. = 15

THIS MECHANISM HAS 6 HINGES EXTERNAL WORK = 162.0000(SF) MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION 3 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 4 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 9 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION 10 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION 15 ANGLE ROTATION = 1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION 16 ANGLE ROTATION = -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4

```
THIS IS ELEMENTARY MECHANISM NO. = 1[illegible]

THIS MECHANISM HAS   6  HINGES   EXTERNAL WORK =  27[illegible](SF)        MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION   1   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   2   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   7   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION   8   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  13   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  14   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4

THIS IS ELEMENTARY MECHANISM NO. =  17

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0    (SF)        MECHANISM CODE =  0

THIS HINGE IS AT SECTION  12   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  21   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  22   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 1

THIS IS ELEMENTARY MECHANISM NO. =  18

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0    (SF)        MECHANISM CODE = [illegible]

THIS HINGE IS AT SECTION   4   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   5   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  25   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 2

THIS IS ELEMENTARY MECHANISM NO. =  19

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.0    (SF)        MECHANISM CODE =  0

THIS HINGE IS AT SECTION  10   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  11   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  27   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  29   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
```

```
THIS IS ELEMENTARY MECHANISM NO. =  20

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION  16   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  17   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  30   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2

THIS IS ELEMENTARY MECHANISM NO. =  21

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION   2   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   3   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  31   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 2

THIS IS ELEMENTARY MECHANISM NO. =  22

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION   8   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION   9   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  33   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  34   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 3

THIS IS ELEMENTARY MECHANISM NO. =  23

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION  14   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  15   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  36   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
```

```
THIS IS ELEMENTARY MECHANISM NO. = 24

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =

THIS HINGE IS AT SECTION  12   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  21   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 1
THIS HINGE IS AT SECTION  22   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 1


THIS IS ELEMENTARY MECHANISM NO. =  25

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION   4   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   5   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  25   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2


THIS IS ELEMENTARY MECHANISM NO. =  26

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION  10   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  11   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  27   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 2
THIS HINGE IS AT SECTION  28   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 2


THIS IS ELEMENTARY MECHANISM NO. =  27

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0   (SF)      MECHANISM CODE =  0

THIS HINGE IS AT SECTION  16   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  17   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  30   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
```

```
THIS IS ELEMENTARY MECHANISM NO. =  28

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0    (SF)        MECHANISM CODE =  0

THIS HINGE IS AT SECTION   2   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION   3   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  31   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3

THIS IS ELEMENTARY MECHANISM NO. =  29

THIS MECHANISM HAS   4  HINGES   EXTERNAL WORK =    0.0    (SF)        MECHANISM CODE =  0

THIS HINGE IS AT SECTION   8   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION   9   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 5
THIS HINGE IS AT SECTION  33   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 3
THIS HINGE IS AT SECTION  34   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 3

THIS IS ELEMENTARY MECHANISM NO. =  30

THIS MECHANISM HAS   3  HINGES   EXTERNAL WORK =    0.0    (SF)        MECHANISM CODE =  0

THIS HINGE IS AT SECTION  14   ANGLE ROTATION =  -1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  15   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 4
THIS HINGE IS AT SECTION  36   ANGLE ROTATION =   1.0000AND PLASTIC MOMENT CAPACITY IS MP 3

      WHERE THE NEGATIVE MECHANISM CODE NUMBER SHOWS A RIGHT-TO-LEFT LATERAL MECHANISMS
```

NONCOMPOSITE STEEL FRAMES

NUMBER OF CONSTRAINS IS 5

NUMBER OF ROWS IN MATRIX A(III,JJ) IS 7

NUMBER OF COLUMNS IN MATRIX A(III,JJ) IS 282

UNBOUNDED

ITERATION	OBJ. FUNCTION	NEW BASIC VAR.
1	17212.49	1
2	34424.99	3
3	51637.48	4
4	59899.48	7
5	59899.48	276
6	65601.89	275
7	69257.36	2
8	73172.89	248
9	74391.34	273
10	74391.34	265
11	74391.35	266

OBJ. FUNCTION 74391.35370

VARIABLE	VALUE	INPUT COST
1	331.16195	60.00000
2	308.02872	60.00000
3	308.02872	60.00000
4	154.01371	72.00000
5	179.70009	36.00000

COMPOSITE STEEL FRAMES

CYCLE NO. 2

ASSUMED RATIO OF MU/MP

MU/MP(1)= 1.000
MU/MP(2)= 1.000
MU/MP(3)= 1.000
MU/MP(4)= 1.000
MU/MP(5)= 1.000
MU/MP(6)= 1.000
MU/MP(7)= 1.000
MU/MP(8)= 1.000
MU/MP(9)= 1.000
MU/MP(10)= 1.000
MU/MP(11)= 1.000
MU/MP(12)= 1.000
MU/MP(13)= 1.000
MU/MP(14)= 1.000
MU/MP(15)= 1.000
MU/MP(16)= 1.000
MU/MP(17)= 1.000
MU/MP(18)= 1.000
MU/MP(19)= 1.000
MU/MP(20)= 1.911
MU/MP(21)= 1.000
MU/MP(22)= 1.000
MU/MP(23)= 1.911
MU/MP(24)= 1.000
MU/MP(25)= 1.000
MU/MP(26)= 1.934
MU/MP(27)= 1.000
MU/MP(28)= 1.000
MU/MP(29)= 1.934
MU/MP(30)= 1.000
MU/MP(31)= 1.000
MU/MP(32)= 1.934
MU/MP(33)= 1.000
MU/MP(34)= 1.000
MU/MP(35)= 1.934
MU/MP(36)= 1.000

NUMBER OF CONSTRAINS IS 5

NUMBER OF ROWS IN MATRIX A(III,JJ) IS 7

NUMBER OF COLUMNS IN MATRIX A(III,JJ) IS 387

UNBOUNDED ITERATION	OBJ. FUNCTION	NEW BASIC VAR.
1	11825.82	1
2	23560.59	3
3	35295.36	4
4	43557.36	7
5	43557.36	381
6	49259.77	380
7	50416.93	2
8	52657.97	378
9	52657.97	370
10	52725.65	371
11	52801.19	347

OBJ. FUNCTION 52801.19033

VARIABLE	VALUE	INPUT COST
1	215.73095	60.00000
2	214.00759	60.00000
3	214.00759	60.00000
4	107.00305	72.00000
5	179.70009	36.00000

COMPOSITE STEEL FRAMES

CYCLE NO. [illegible]

ASSUMED RATIO OF MU/MP

```
MU/MP(  1)=  1.000
MU/MP(  2)=  1.000
MU/MP(  3)=  1.000
MU/MP(  4)=  1.000
MU/MP(  5)=  1.000
MU/MP(  6)=  1.000
MU/MP(  7)=  1.000
MU/MP(  8)=  1.000
MU/MP(  9)=  1.000
MU/MP( 10)=  1.000
MU/MP( 11)=  1.000
MU/MP( 12)=  1.000
MU/MP( 13)=  1.000
MU/MP( 14)=  1.000
MU/MP( 15)=  [illegible]
MU/MP( 16)=  [illegible]
MU/MP( 17)=  1.000
MU/MP( 18)=  1.000
MU/MP( 19)=  1.000
MU/MP( 20)=  2.054
MU/MP( 21)=  1.000
MU/MP( 22)=  1.000
MU/MP( 23)=  2.054
MU/MP( 24)=  1.000
MU/MP( 25)=  1.000
MU/MP( 26)=  2.057
MU/MP( 27)=  1.000
MU/MP( 28)=  1.000
MU/MP( 29)=  2.057
MU/MP( 30)=  1.000
MU/MP( 31)=  1.000
MU/MP( 32)=  2.057
MU/MP( 33)=  1.000
MU/MP( 34)=  1.000
MU/MP( 35)=  2.057
MU/MP( 36)=  1.000
```

NUMBER OF CONSTRAINS IS 5

NUMBER OF ROWS IN MATRIX A(III,JJ) IS 7

NUMBER OF COLUMNS IN MATRIX A(III,JJ) IS 413

UNBOUNDED

ITERATION	OBJ. FUNCTION	NEW BASIC VAR.
1	11270.79	1
2	22530.51	3
3	33790.23	4
4	42052.22	7
5	42052.22	407
6	47754.63	406
7	50726.70	404
8	50888.52	2
9	50888.52	396

OBJ. FUNCTION 50888.41832

VARIABLE	VALUE	INPUT COST
1	204.23942	60.00000
2	208.20006	60.00000
3	232.96241	50.00000
4	104.10003	72.00000
5	179.70009	36.00000

COMPOSITE STEEL FRAMES

CYCLE NO. 4

ASSUMED RATIO OF MU/MP

MU/MP(1)=	1.000
MU/MP(2)=	1.000
MU/MP(3)=	1.000
MU/MP(4)=	1.000
MU/MP(5)=	1.000
MU/MP(6)=	1.000
MU/MP(7)=	1.000
MU/MP(8)=	1.000
MU/MP(9)=	1.000
MU/MP(10)=	1.000
MU/MP(11)=	1.000
MU/MP(12)=	1.000
MU/MP(13)=	1.000
MU/MP(14)=	1.000
MU/MP(15)=	1.000
MU/MP(16)=	1.000
MU/MP(17)=	1.000
MU/MP(18)=	1.000
MU/MP(19)=	1.000
MU/MP(20)=	2.075
MU/MP(21)=	1.000
MU/MP(22)=	1.000
MU/MP(23)=	2.075
MU/MP(24)=	1.000
MU/MP(25)=	1.000
MU/MP(26)=	2.067
MU/MP(27)=	1.000
MU/MP(28)=	1.000
MU/MP(29)=	2.067
MU/MP(30)=	1.000
MU/MP(31)=	1.000
MU/MP(32)=	2.077
MU/MP(33)=	1.000
MU/MP(34)=	1.000
MU/MP(35)=	2.077
MU/MP(36)=	1.000

NUMBER OF CONSTRAINS IS 5

NUMBER OF ROWS IN MATRIX A(III,JJ) IS 7

NUMBER OF COLUMNS IN MATRIX A(III,JJ) IS 413

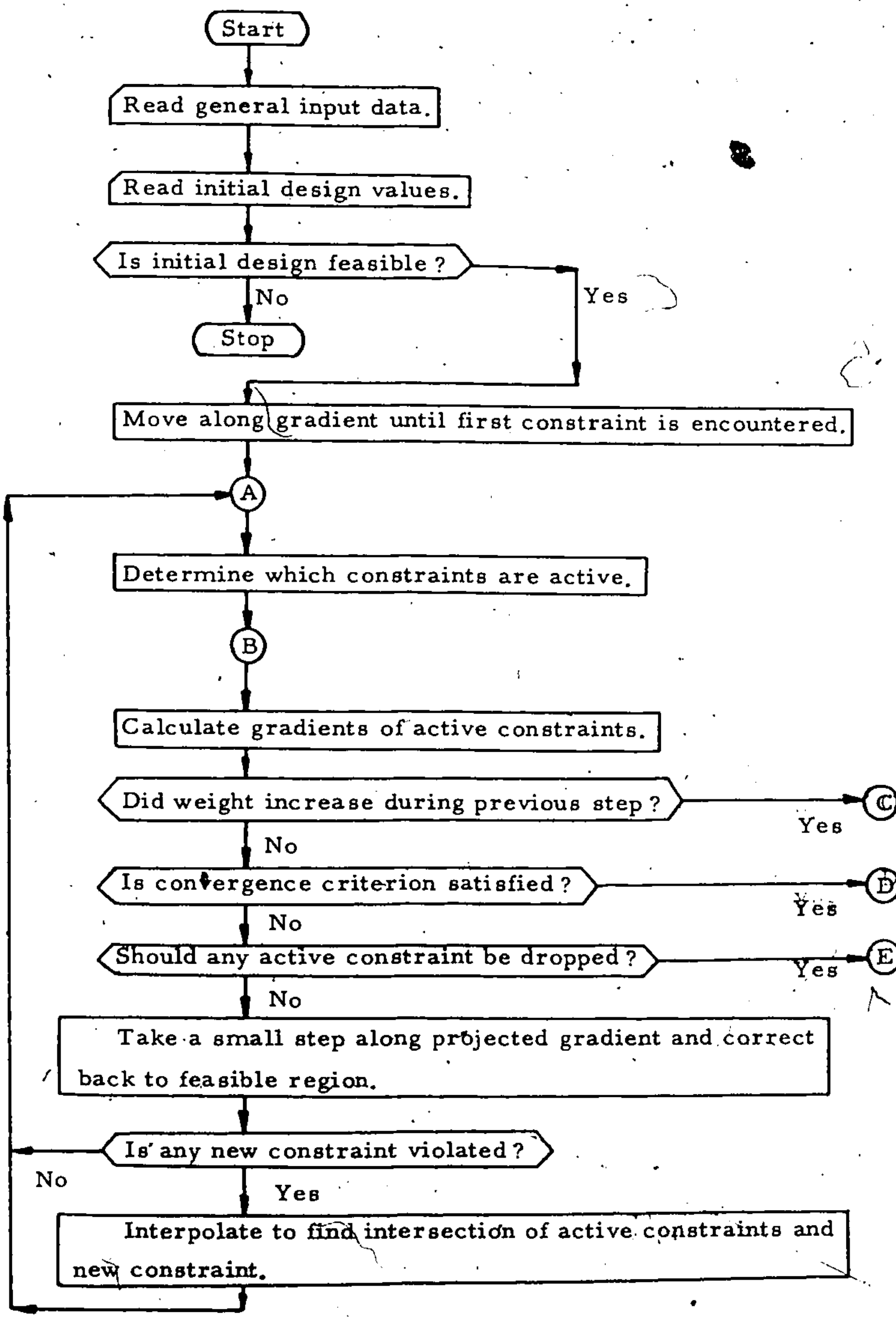
UNBOUNDED ITERATION	OBJ. FUNCTION	NEW BASIC VAR.
1	11196.74	1
2	22419.43	2
3	33607.58	4
4	41869.67	7
5	41869.67	407
6	47572.08	404
7	50513.25	404
8	50670.22	7
9	50672.78	397

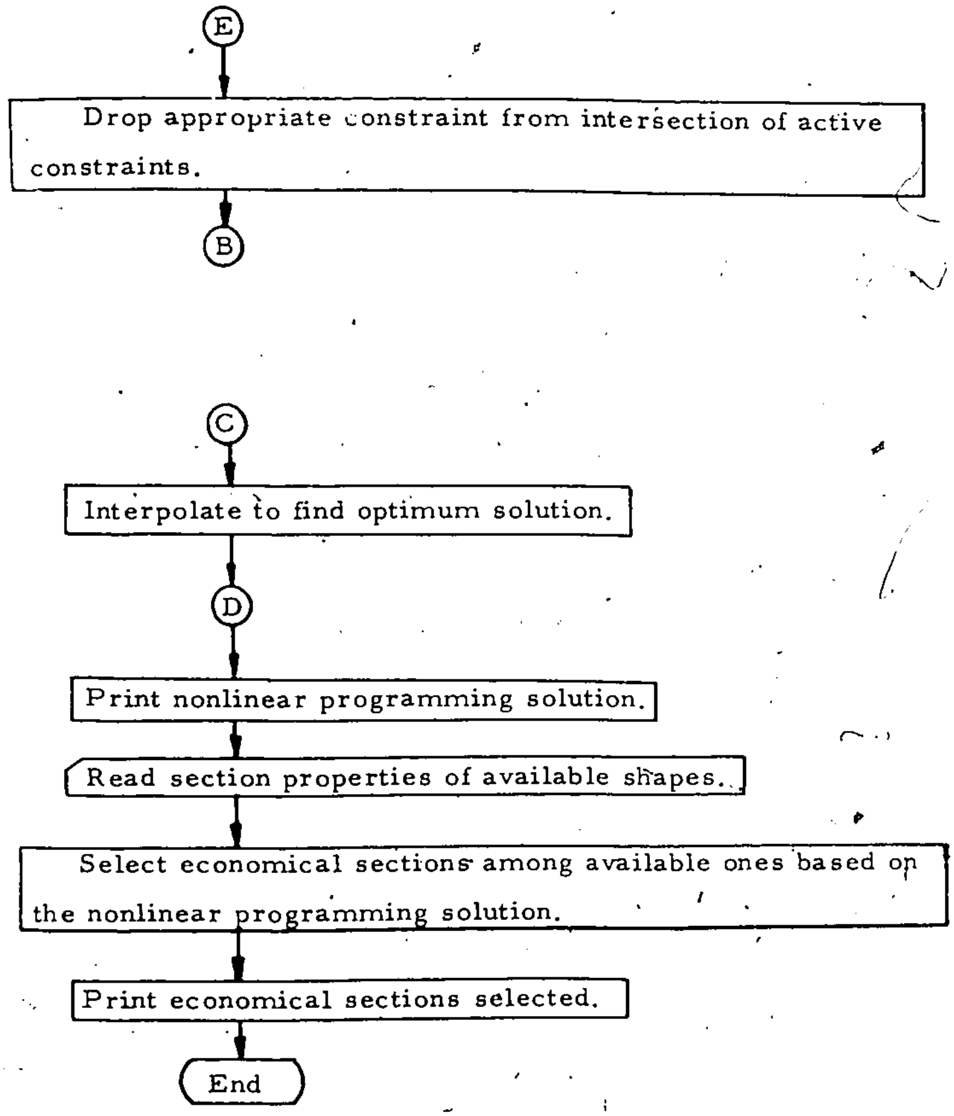
OBJ. FUNCTION 50672.78300

VARIABLE	VALUE	INPUT COST
1	202.68692	60.00000
2	230.95932	60.00000
3	203.21006	66.00000
4	104.17003	72.00000
5	179.70509	36.00000

	MP(NON-COMP.) (K.FT)	MP(NON-COMP.) (WT2.L)	MP(COMP.) (K.FT)	MP(COMP.) (WT2.L)	
1	331.162	0.07215	202.637	0.04415	0.61190...BEAM MEMBER
2	308.029	0.06711	200.969	0.04378	0.65244...BEAM MEMBER
3	308.029	0.06711	208.200	0.04536	0.67531...BEAM MEMBER
4	154.014	0.03355	104.100	0.02268	0.67591...COLUMN MEMBER
5	179.700	0.03915	179.700	0.03915	1.00000...COLUMN MEMBER

A. 4-5 Flow Chart of Main Program for Nonlinear Optimization of Steel Frames including Composite Ones




E
Drop appropriate constraint from intersection of active constraints.
B
C
Interpolate to find optimum solution.
D
Print nonlinear programming solution.
Read section properties of available shapes.
Select economical sections among available ones based on the nonlinear programming solution.
Print economical sections selected.
End

A. 4-6 Descriptions of Auxiliary Subroutines for Nonlinear Optimization of Steel Frames including Composite Ones

INVERT : A subroutine which calculates the inversion of a symmetric matrix $[U_q^T(X)\,U_q(X)]$ which are composed of the gradient of the constraint function a_j, $j = 1, \ldots, m$ at the current design variables.

CONST: A subroutine which calculates the value of the constraint functions on the basis of the current design variables.

OBJECT: A subroutine which computes the values of the objective function and the gradient vector on the basis of the current design variables.

A. 4-7 Descriptions of Input Data for Nonlinear Optimization of Steel Frames including Composite Ones

1) First Set of Data

M, N, NOAV

Where, M, N and NOAV denote the number of constraints, number of variables and number of available rolled sections, if economical selection of sections on the basis of the nonlinear optimum solution, is required, otherwise leave blank.

2) Second Set of Data

EPS, DELTA, XOO, WOO

Where EPS and DELTA denote the tolerance ε and δ. XOO and WOO are the reference values to be used for defining the non-dimensional design variables and objective function, respectively.

3) Third Groups of Data

| BU (i), i = 1, N |

| BL (i), i = 1, N |

Where BU (i) and BL (i) denote the upper and lower bounds for the i-th variable, respectively.

4) Fourth Set of Data

| FL (i), i = 1, N |

Where FL (i) denotes the i-th member's length. If an economical selection of members from available rolled sections is required, a member length for each column must be given in negative value for a convenience in identifying a column member from beam members.

5) Fifth Set of Data

| X (i), i = 1, N |

In which the initial value of the i-th design variable X (i) must be assumed.

6) Sixth and Last Groups of Data

| IAVDP (i), AVWT (i), AVPM (i) |

... for i = 1, NOAV

Where IAVDP (i), AVWT (i) and AVPM (i) denote the nominal depth, weight and plastic moment capacity of the i-th available shape of a rolled wide flange section or a light beam or a standard beam. If IAVDP (i) corresponds to a rolled wide flange, it must be given in positive integer, otherwise must be in negative integer in order to identify wide flanges from other shapes since for column members only wide flanges are to be used.

A. 4-8 List of Programs for Nonlinear Optimization of Steel Frames including Composite Ones

```
//UOCC1956    JOB    (
//            1,RAMA               ',ROELMV 1=( ,0),                              J...
//            CLASS=B
//      EXEC FORTGCLG,REGION.FORT=150K,REGION.GO=100K,TIME.GO=5
//FORT.SYSIN DD *
IEF142I - STEP WAS EXECUTED - COND CODE 0000
IEF373I STEP /FORT     / START 72333.2123
IEF374I STEP /FORT     / STOP  72333.2124 CPU   0MIN 41.30SEC MAIN 148K LCS    OK
IEF142I - STEP WAS EXECUTED - COND CODE 0000
IEF373I STEP /LKED     / START 72333.2124
IEF374I STEP /LKED     / STOP  72333.2125 CPU   0MIN 03.02SEC MAIN 100K LCS    OK
//GO.SYSIN DD *
//SYSIN     DD *             GENERATED STATEMENT
IEF142I - STEP WAS EXECUTED - COND CODE 0000
IEF373I STEP /GO       / START 72333.2125
IEF374I STEP /GO       / STOP  72333.2125 CPU   0MIN 18.20SEC MAIN  66K LCS    OK
IEF375I JOB /UOCC1956/ START 72333.2123
IEF376I JOB /UOCC1956/ STOP  72333.2125 CPU   1MIN 02.61SEC


*OPTIONS IN EFFECT*  ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,NOMAP
*OPTIONS IN EFFECT*  NAME = MAIN   , LINECNT =      45
*STATISTICS*    SOURCE STATEMENTS =       563,PROGRAM SIZE =    23264
*STATISTICS*  NO DIAGNOSTICS GENERATED

*OPTIONS IN EFFECT*  ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,NOMAP
*OPTIONS IN EFFECT*  NAME = INVERT , LINECNT =      45
*STATISTICS*    SOURCE STATEMENTS =        43,PROGRAM SIZE =     1490
*STATISTICS*  NO DIAGNOSTICS GENERATED

*OPTIONS IN EFFECT*  ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,NOMAP
*OPTIONS IN EFFECT*  NAME = CONST  , LINECNT =      45
*STATISTICS*    SOURCE STATEMENTS =       107,PROGRAM SIZE =    10132
*STATISTICS*  NO DIAGNOSTICS GENERATED

*OPTIONS IN EFFECT*  ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,NOMAP
*OPTIONS IN EFFECT*  NAME = OBJECT , LINECNT =      45
*STATISTICS*    SOURCE STATEMENTS =        16,PROGRAM SIZE =      904
*STATISTICS*  NO DIAGNOSTICS GENERATED

*STATISTICS*  NO DIAGNOSTICS THIS STEP 2
```

```
      C          NONLINEAR PROGRAMMING APPROACH TO THE MINIMUM WEIGHT PLASTIC
      C          DESIGN OF RIGID STRUCTURES
      C          PROGRAMMED BY [illegible] AT UNIVERSITY OF OTTAWA,FEB,[illegible]
      C
0001             DIMENSION X(10),X0(10),P(40),PP(40),PA(40),PU(10),PL(10),G1(40),
               1           G(40),U(10,40),UQ(10,40),WQ(40),V2(40,40),R(40),R1(40),
               2           B(40),PQG(40),FL(10),Z(10),COEF1(40),COEF(10),
               3           WT(10),PM(10),IPM(10),
               4           PM1(10),WT1(10),IP1(10),PM2(10),WT2(10),IP2(10),
               5           IAVOP(100),AVWT(100),AVPM(100)
0002             NREAD=1
0003             NWRIT=3
      C
      C          READ IN THE INPUT DATA  REQUIRED IN MAINPROGRAM
      C          READ IN THE NUMBER OF CONSTRAINTS M
      C          READ IN THE NUMBER OF VARIABLES N
      C          READ IN THE NUMBER OF AVAILABLE SECTIONS, IF AVAILABLE SOL,N PRO,D
      C
0004             READ(NREAD,10) M,N,NOAV
0005          10 FORMAT(20I4)
      C
      C          READ IN THE TOLERANCES OF EPS AND DELTA
      C          READ IN THE REFERANCE VALUES OF XOO AND WOO
      C
0006             READ(NREAD,15) EPS,DELTA,XOO,WOO
0007          15 FORMAT(8F10.0)
      C
      C          READ IN THE UPPER AND LOWER BOUNDS
      C
0008             READ(NREAD,15) (PM(I),I=1,N)
0009             READ(NREAD,15) (PL(I),I=1,N)
      C
      C          READ IN THE MEMBER LENGTHS
      C          & IF AVAILABLE SOL. REQUIRED,PUT NEGATIVE SIGN FOR COLUMN MEMBER
      C          FOR IDENTIFING AS A COLUMN MEMBER IN SORTING AVAILABLE COLUMN SEC.
      C
0010             READ(NREAD,15) (FL(I),I=1,N)
      C
      C          PRINT INPUT DATA
      C
0011             WRITE(NWRIT,20) M,N,EPS,DELTA,XOO,WOO
0012          20 FORMAT(1H1//10X,'THE NUMBER OF CONSTRAINT FUNCTION M=',I4
```

```
     1//10X,'THE NUMBER OF DESIGN VARIABLES N=',I6,
     2//10X,'THE TOLERANCE DELT=',E15.7,2X,
     3'  ( WHICH CONTROLS THE NEARNESS OF THE POINT XMAX TO THE TRUE OPT
     4IMUM POINT )',
     5//10X,'THE TOLERANCE DELTA=',E15.7,
     6'  ( WHICH CONTROLS THE NEARNESS OF THE POINT XV TO THE INTERSECT
     7ION S )',
     8//10X,'THE REFERANCE VALUE XOO=',F10.0,
     8'  ( USED FOR DEFINING THE NON-DIMENSIONAL DESIGN VARIABLES )'
     9//10X,'THE REFERANCE VALUE WOO=',F10.0,
     9'  ( USED FOR DEFINING THE NON-DIMENSIONAL OBJECTIVE FUNCTION )'
     9///)
C
C     SET  THE MEMBER CODE TO IDENTIFY WHETHER THE MEMBER IS A BEAM
C     OR A COLUMN
C
      DO 30 I=1,N
      IEM(I)=1
      IF (FL(I)) 25,30,30
   25 FL(I)=-FL(I)
      IEM(I)=-1
   30 CONTINUE
      WRITE(NWRIT,35)
   35 FORMAT(//10X,'THE MEMBER LENGTH    MEMBER CODE'/10X,'L',11X,
     1'( WHERE NEGATIVE AND POSITIVE CODES SHOW COLUMN AND BEAM MEMBERS
     2RESPECTIVELY )'/)
      DO 40 I=1,N
   40 WRITE(NWRIT,45) I,FL(I),IEM(I)
   45 FORMAT(I10,F15.5,I12)
C
      LLLL=0
C
C     READ IN THE INITIAL VALUES OF THE DESIGN VARIABLES
C
   60 READ(NREAD,15) (X(I),I=1,N)
      LLLL=LLLL+1
      SUM=0.
      DO 61 I=1,N
   61 SUM=SUM+X(I)
      IF (SUM) 9999,9999,62
   62 WRITE(NWRIT,63)
   63 FORMAT(1H1)
      WRITE(NWRIT,64)
```

```
0034       66 FORMAT(//5X,'THE INITIAL [illegible] VA[illegible]ABLES XA [illegible]ALIZED
             1VARIABLE X, [illegible] FOLLOWS'//
             210X,'X[illegible]',1[illegible]X,'X',1[illegible]X,'BU',[illegible]X,'BL'/)
0035          DO 66 I=1,N
0036          TN=X(I)
0037          X(I)=X(I)/XOO
0038       66 WRITE(NWRIT,67) I,TN,X(I),BU(I),BL(I)
0039       67 FORMAT(I10,F15.5,3F15.6)
     C
     C        SET THE MAXIMUM LENGTH OF FEASIBLE REGION R
     C
0040          SUM=0.
0041          DO 69 I=1,N
0042       69 SUM=SUM+(BU(I)+BL(I))**2
0043          FLR=SQRT(SUM)
     C
     C        SET A STEP LENGTH AND OTHER VALUES
     C
0044          STEP=FLR/100.
0045          DELTA2=DELTA*DELTA
0046          WO=10.**20
0047          FN=N
     C
     C        CALCULATE THE VALUES OF THE CONSTRAINT FUNCTION PHI(I),I=1,....,M
     C        ON THE BASIS OF THE CURRENT DESIGN VARIABLES
     C
0048          CALL CONST(P,X,M,N,XOO)
     C
     C        CHECK WETHER THE INITIAL POINT IS AN INTERIOR OF R OR NOT.
     C        CHECK CONSTRAINTS WHETHER ACTIVE OR NOT
     C
0049          PMIN=P(1)
0050          DO 75 I=2,M
0051          IF (PMIN-P(I)) 75,75,70
0052       70 PMIN=P(I)
0053       75 CONTINUE
0054          IF (PMIN) 80,80,95
0055       80 IF (PMIN*PMIN-DELTA2) 250,250,85
0056       85 WRITE(NWRIT,90)
0057       90 FORMAT(1H1//10X,'INITIAL TRIAL VALUES ARE NOT GOOD')
0058          GO TO 40
0059       95 IF (PMIN*PMIN-DELTA2) 250,250,100
     C
```

```
C        AT CONSTRAINTS ARE ACTIVE, MOVE ALONG GRADIENT UNTIL
C        FIRST CONSTRAINT IS ENCOUNTERED
C        USE ONE-DIMENSIONAL INTERPOLATION METHOD
C
C
C        COMPUTE THE VALUES OF THE OBJECTIVE FUNCTION AND THE GRADIENT
C        VECTOR ON THE BASIS OF THE CURRENT DESIGN VARIABLES
C
     100 CALL OBJECT (F,G,X,FL,WT,N,XOO,WOO)
         TM=-F
         W=-F*WOO
         WRITE(NWRIT,105) TM
     105 FORMAT(//10X,'W/WOO=',F15.5)
         SUM=0.
         DO 110 I=1,N
     110 SUM=SUM+G(I)*G(I)
         DO 115 I=1,N
     115 G1(I)=G(I)/SQRT(SUM)
         T=STEP
         PMIN=P(1)
         DO 125 I=2,M
         IF (PMIN-P(I)) 125,125,120
     120 PMIN=P(I)
     125 CONTINUE
         DO 130 I=1,N
     130 XO(I)=X(I)
         PMINO=PMIN
     140 TO=T
         DO 145 I=1,N
     145 X(I)=X(I)+T*G1(I)
         DO 160 I=1,N
         IF (X(I)) 150,150,160
     150 DO 155 J=1,N
     155 X(J)=XO(J)
         T=T/2.
         GO TO 140
     160 CONTINUE
         DO 165 I=1,N
     165 WRITE(NWRIT,170) I,X(I)
     170 FORMAT (10X,'X(',I4,')=',E15.7)
         CALL CONST(P,X,M,N,XOO)
         DO 172 I=1,M
     172 WRITE(NWRIT,173) I,P(I)
```

```
  173 FORMAT (11X,I2,(' [illegible] ')=',E15.7)
      DO 175 I=1,M
      IF (P(I)) 180,175,175
  175 CONTINUE
      GO TO 100
  180 PMIN=P(1)
      DO 190 I=2,M
      IF (PMIN-P(I)) 190,190,185
  185 PMIN=P(I)
  190 CONTINUE
  200 TM=PMINO-PMIN
      IF (ABS(TM)-1./10.**20) 202,202,204
  202 DELTA=ABS(PMIN)
      DELTA2=DELTA*DELTA
      WRITE(NWRIT,203) DELTA
  203 FORMAT(/10X,'UNABLE TO SATISFY THE TOLERANCE DELTA, HENCE RESET DE
     1LTA TO ',E15.7)
      GO TO 250
  204 T=TO*(PMIN/(PMINO-PMIN))
      DO 205 I=1,N
  205 XO(I)=X(I)
      PMINO=PMIN
  210 TO=T
      DO 215 I=1,N
  215 X(I)=X(I)+T*GI(I)
      DO 230 I=1,N
      IF (X(I)) 220,230,230
  220 DO 225 J=1,N
  225 X(J)=XO(J)
      T=T/2.
      GO TO 210
  230 CONTINUE
      DO 235 I=1,N
  235 WRITE(NWRIT,170) I,X(I)
      CALL CONST(P,X,M,N,XOO)
      DO 237 I=1,M
  237 WRITE(NWRIT,173) I,P(I)
      PMIN=P(1)
      DO 245 I=2,M
      IF (PMIN-P(I)) 245,245,240
  240 PMIN=P(I)
  245 CONTINUE
      IF (PMIN*PMIN-DELTA2) 250,250,200
```

```
  250 CONTINUE
C
C     DETERMINE WHICH CONSTRAINTS ARE ACTIVE
C
      ERR=0.
      IQ=0
      DO 255 I=1,M
  255 PA(I)=0.
  260 PMIN=10.**20
      DO 275 I=1,M
      IF (PA(I)) 265,265,275
  265 IF (P(I)-PMIN) 270,270,275
  270 PMIN=P(I)
      IFOUND=I
  275 CONTINUE
      ERR=ERR+P(IFOUND)*P(IFOUND)
      IF (ERR-DELTA2) 280,280,285
  280 IQ=IQ+1
      PA(IFOUND)=1.
      GO TO 260
  285 CONTINUE
C
C     CALCULATE GRADIENTS OF ALL CONSTRAINTS
C
      DO 290 I=1,N
  290 XO(I)=X(I)
      DO 295 I=1,M
  295 PO(I)=P(I)
      DO 300 I=1,N
  300 WRITE(NWRIT,170) I,X(I)
      WRITE(NWRIT,305) IQ
  305 FORMAT(//10X,'Q =',I4//)
      DO 310 I=1,M
  310 WRITE(NWRIT,315) I,PA(I),I,P(I)
  315 FORMAT (10X,'PHI-ACTIVE (',I4,' )=',F3.0,15X,'PHI (',I4,' )=',
     1E15.7)
      DO 330 I=1,N
      DO 320 J=1,M
  320 L(I,J)=0.
      DO 325 I=1,N
      DO 325 J=1,N
  325 X(J)=XO(J)
      X(I)=X(I)+X(I)/100.
```

```
      CALL CONST(X,G,M,IX)
      DO 330 J=1,M
  330 U(I,J)=(P(J)-PC(J))/(X(I)/100.)
  335 CONTINUE
      WRITE(NWRIT,336)
  336 FORMAT(/' GRADIENT MATRIX U(N,M)...GRADIENTS OF ALL CONSTRAINTS')
      DO 338 I=1,N
      WRITE(NWRIT,337) I
  337 FORMAT(/2X,' ROW ',I2)
  338 WRITE(NWRIT,339) (U(I,J),J=1,M)
  339 FORMAT(5X,8E15.7)
      DO 340 I=1,N
  340 X(I)=XO(I)
      DO 345 I=1,M
  345 P(I)=PC(I)
  350 CONTINUE
C
C     CALCULATE GRADIENTS OF ACTIVE CONSTRAINTS
C
      J=1
      DO 365 IP=1,M
      IF (PA(IP)) 365,365,355
  355 VC(J)=P(IP)
      DO 360 I=1,N
  360 UO(I,J)=U(I,IP)
      J=J+1
  365 CONTINUE
      WRITE(NWRIT,366)
  366 FORMAT(/' ACTIVE GRADIENT MATRIX UO(N,Q)...GRADIENTS OF ACTIVE CON
     1STRAINTS (M>Q)')
      DO 367 I=1,N
      WRITE(NWRIT,337) I
  367 WRITE(NWRIT,339) (UO(I,J),J=1,IQ)
      DO 375 I=1,IQ
      DO 375 J=I,IQ
      VO(I,J)=0.
      DO 370 K=1,N
  370 VO(I,J)=VO(I,J)+UO(K,I)*UO(K,J)
      VO(J,I)=VO(I,J)
  375 CONTINUE
      WRITE(NWRIT,380)
  380 FORMAT(/' (UO)T*(UO) MATRIX')
      DO 385 I=1,IQ
```

```
      WRITE(NWRIT,337) I
  335 WRITE(NWRIT,333) (VO(I,J),J=1,IO)
      CALL OBJECT (F,G,X,H,WT,W,YO,XP)
      WRITE(NWRIT,405)
  405 FORMAT(/' VO MATRIX')
      DO 410 I=1,IO
      WRITE(NWRIT,337) I
  410 WRITE(NWRIT,333) (VO(I,J),J=1,IO)
      DO 420 I=1,IO
  420 WRITE(NWRIT,425) I,WO(I)
  425 FORMAT (10X,'WO (',I4,' )=',E15.7)
      TM=-F
      W=-F*WOP
      WRITE(NWRIT,105) TM
C
C     DID WEIGHT INCREASE DURING PREVIOUS STEP
C
C     THUS THE LOCAL MINIMUM VALUES ARE ASSUMED TO BE GLOBAL OPTIMUM
C     VALUES, AND SO REPEAT THE WHOLE CALCULATIONS FOR DIFFERENT INITIAL
C     VARIABLES TO GET THE REAL RESULTS.
C
      TM=WO-W
      WRITE(NWRIT,440) TM
  440 FORMAT(/10X,'WOLD-W=',E15.7)
      IF (WO-W) 900,450,450
  450 WO=W
      IF (IO-1) 452,452,453
  452 VO(1,1)=1./VO(1,1)
      GO TO 454
  453 CALL INVERT(VO,IO)
  454 DO 460 I=1,IO
      R1(I)=0.
      DO 455 J=1,N
  455 R1(I)=R1(I)+UO(J,I)*G(J)
  460 CONTINUE
      DO 470 I=1,IO
      R(I)=0.
      DO 465 J=1,IO
  465 R(I)=R(I)+VO(I,J)*R1(J)
  470 CONTINUE
      DO 480 I=1,N
      POG(I)=G(I)
      DO 475 J=1,IO
```

```
      PQG(I)=PQG(I)-B(I,J)*X(J)
  475 CONTINUE
  480 CONTINUE
      DO 485 I=1,IQ
  485 B(I)=B(I)/(2.*SQRT(VQ(I,I)))
      BMAX=B(1)
      IDROP=1
      IF (IQ-1) 497,497,487
  487 DO 495 I=2,IQ
      IF (BMAX-B(I)) 490,495,495
  490 BMAX=B(I)
      IDROP=I
  495 CONTINUE
  497 SUM=0.
      DO 500 I=1,N
  500 SUM=SUM+PQG(I)*PQG(I)
      ABPQG=SQRT(SUM)
      WRITE(NWRIT,502) ABPQG,B(IDROP)
  502 FORMAT(/10X,'ABS(PQG)=',E15.7,'          B(IDROP)=',E15.7)
C
C     SET BET TO THE BIGGER VALUE OF ABS(PQG) AND B(IDROP)
C
      IF (ABPQG-B(IDROP)) 505,510,510
  505 BET=B(IDROP)
      GO TO 515
  510 BET=ABPQG
  515 WRITE(NWRIT,520) BET
  520 FORMAT(//10X,'BET =',E15.7)
C
C     IS COVERGENCE CRITERION SATISFIED
C
      IF (BET-EPS/(2.*EN*EL2)) 910,910,525
  525 IF (ABPQG-B(IDROP)) 530,570,570
C
C     DROP APPROPRIATE CONSTRAINT FROM INTERSECTION OF ACTIVE CONSTRAT.
C
  530 J=1
      DO 555 I=1,M
      IF (PA(I)) 555,555,535
  535 IF (J-IDROP) 550,540,550
  540 PA(I)=0.
      IQ=IQ-1
      WRITE(NWRIT,545) IDROP,IQ
```

```
575 FORMAT(//10X,'CYCLE ',I3,' IS',' F=',I4)
    GO TO 565
552 J=J+1
555 CONTINUE
    WRITE(NWRIT,560)
560 FORMAT(//10X,'ERROR-0 ... ERRORS/')
    GO TO 9999
565 DO 569 I=1,M
569 WRITE(NWRIT,315) I,RA(I),I,P(I)
    GO TO 350
570 CONTINUE
C
C     TAKE A SMALL STEP ALONG PROJECTED GRADIENT AND CORRECT BACK TO
C     FEASIBLE REGION
C
C
C     ITERATION TO OBTAIN FEASIBLE POINT X(V+1) IN INTERSECTION G
C
    WRITE(NWRIT,572)
572 FORMAT(/10X,'ITERATION PROCEDURE IS REQUIRED')
    DO 575 I=1,N
    Z(I)=PGS(I)/ABPGS
575 XO(I)=X(I)
    DO 580 I=1,M
580 PO(I)=P(I)
    TAU=STEP
585 DO 590 I=1,N
590 X(I)=X(I)+TAU*Z(I)
    DO 605 I=1,N
    IF (X(I)) 595,605,605
595 DO 600 J=1,N
600 X(J)=XO(J)
    TAU=TAU/2.
    GO TO 585
605 CONTINUE
    DO 607 I=1,N
607 WRITE(NWRIT,608) I,Z(I),I,PGS(I)
608 FORMAT (10X,'UNIT VECTOR Z (',I4,')=',F15.7,10X,'PGS (',I4,
   1')=',F15.7)
    DO 610 I=1,N
610 WRITE(NWRIT,170) I,X(I)
620 CONTINUE
```

```
      [illegible]
      J=1
      DO 430 I=1,N
      IF (DA(I)) 430,425,425
  425 WD(J)=D(I)
      J=J+1
  430 CONTINUE
      ERR=0.
      DO 435 I=1,IC
  435 ERR=ERR+WD(I)*WD(I)
      IF (ERR-DELTA2) 480,480,440
  440 DO 450 I=1,IC
      CORR1(I)=0.
      DO 445 J=1,IC
  445 CORR1(I)=CORR1(I)+V2(I,J)*WD(J)
  450 CONTINUE
      DO 460 I=1,N
      CORR(I)=0.
      DO 455 J=1,IC
  455 CORR(I)=CORR(I)+UD(I,J)*CORR1(J)
  460 CONTINUE
      DO 465 I=1,N
  465 X(I)=X(I)-CORR(I)
      DO 467 I=1,IC
  467 WRITE(NWRIT,475) I,WD(I)
      DO 470 I=1,N
  470 WRITE(NWRIT,476) I,X(I),I,CORR(I)
  476 FORMAT (10X,'X(',I4,')=',E15.7,5X,'CORR (',I4,')=',E15.7)
      GO TO 420
  480 DO 690 I=1,N
      IF (DA(I)) 685,685,690
C
C     IS ANY NEW CONSTRAINT VIOLATED
C
  685 IF (D(I)) 700,690,690
  690 CONTINUE
      DO 695 I=1,N
  695 WRITE(NWRIT,170) I,X(I)
      GO TO 250
C
C     INTERPOLATE TO FIND INTERSECTION OF ACTIVE CONSTRAINT
```

```
  700 T=T+1
      II=T
      WRITE(NWRIT,702)
  702 FORMAT(/10X,'INTERPOLATION PROCEDURE IS REQUIRED')
      GO TO 900
  705 CONTINUE
      DO 710 I=1,N
  710 X(I)=X(I)+T*Z(I)
      DO 715 I=1,N
  715 WRITE(NWRIT,170) I,X(I)
C
C     TAKE CORRECTION PROCEDURE
C
  720 CONTINUE
      CALL CONST(P,X,M,N,XOC)
      J=1
      DO 735 I=1,M
      IF (PA(I)) 735,735,730
  730 WO(J)=P(I)
      J=J+1
  735 CONTINUE
      ERR=0.
      DO 740 I=1,IC
  740 ERR=ERR+WO(I)*WO(I)
      IF (ERR-DELTA2) 900,900,745
  745 DO 755 I=1,IC
      CORR1(I)=0.
      DO 750 J=1,IC
  750 CORR1(I)=CORR1(I)+VO(I,J)*WO(J)
  755 CONTINUE
      DO 765 I=1,N
      CORR(I)=0.
      DO 760 J=1,IO
  760 CORR(I)=CORR(I)+UO(I,J)*CORR1(J)
  765 CONTINUE
      DO 770 I=1,N
  770 X(I)=X(I)-CORR(I)
      DO 773 I=1,IC
  773 WRITE(NWRIT,425) I,WO(I)
      DO 775 I=1,N
  775 WRITE(NWRIT,675) I,X(I),I,CORR(I)
```

```
      ...
  901 EMIN=10.**...
      ERR1=0.
      DO 815 I=1,N
      IF (PA(I)) 805,805,812
  805 IF (P(I)-PMIN) 810,810,815
  810 PMIN=P(I)
      IADDED=I
      GO TO 815
  812 ERR1=ERR1+P(I)*P(I)
  815 CONTINUE
      WRITE(NWRIT,820) IADDED
  820 FORMAT(//10X,'IADDED=',I4)
      ERR=ERR+PMIN*PMIN
      ERR1=ERR1+PMIN*PMIN
      IF (ERR-DELTA2) 250,250,825
C
C     IF INTERSECTION OF A ACTIVE CONSTRAINT AND NEW CONSTRAINT IS
C     FOUND , GO BACK TO PREVIOUS PROCEDURE . OTHERWISE CONTINUE THE
C     INTERPOLATION PROCEDURE.
C
  825 T1=T1/2.
      DO 850 I=1,N
      IF (X(I)-(X(I)-T1*Z(I))) 830,850,830
  830 DO 832 J=1,N
  832 X(J)=X0(J)
      IF (PMIN) 835,835,840
  835 T=T-T1
      GO TO 705
  840 T=T+T1
      GO TO 705
  850 CONTINUE
      WRITE(NWRIT,855)
  855 FORMAT(//10X,'UNABLE TO ADD NEXT CONSTRAINT')
      DELTA=SQRT(ERR1)
      DELTA2=ERR1
      WRITE(NWRIT,203) DELTA
      GO TO 250
  900 WRITE(NWRIT,905)
  905 FORMAT(//10X,'INCREASE IN OBJECT FUNCTION')
  910 WRITE(NWRIT,911)
  911 FORMAT(1H1//10X,'MINIMUM WEIGHT DESIGN')
      CALL OBJECT (F,G,X,FL,WT,N,XCC,WOO)
```

```
      DO 912 I=1,N
      TM=X(I)*XCO
  912 WRITE(NWRIT,913) I,TM
  913 FORMAT(//10X,'MP',I3,' =',F20.5)
      TWT=-F*WCO
      WRITE(NWRIT,915) TWT
  915 FORMAT(//10X,'TOTAL WEIGHT =',F20.5)
      IF (NOAV) 60,60,918
C
C     SELECT AVAILABLE SECTIONS OF GIVEN WIDE FLANGES
C
  918 OBEN=-F
      DO 919 I=1,N
  919 FM(I)=X(I)*XCO
C
C     READ SECTION PROPERTIES FOR AVAILABLE WF AND B SHAPES
C     WHERE PUT A NEGATIVE SIGN FIRST FOR THE BEAM SHAPES FOR IDENTIFING
C     FROM WIDE FRANGES
C
      IF (LLLL-1) 920,920,955
  920 WRITE (NWRIT,921)
  921 FORMAT(1H1//10X,'AVAILABLE SECTION PROPERTIES'//10X,
     1'( * AVAILABLE ONLY FOR BEAM MEMBERS)'///
     28X,'NO     SHAPES        MP(FT.KIPS)'//)
      DO 950 I=1,NOAV
      READ (NREAD,925) IAVDP(I),AVWT(I),AVPM(I)
  925 FORMAT(I3,2X,F4.0,F6.1)
      IF (IAVDP(I)) 930,940,940
  930 J=-IAVDP(I)
      WRITE(NWRIT,935) I,J,AVWT(I),AVPM(I)
  935 FORMAT(I10,I4,'* B',F5.1,F12.1)
      GO TO 950
  940 WRITE(NWRIT,945) I,IAVDP(I),AVWT(I),AVPM(I)
  945 FORMAT(I10,I4,' WF',F5.1,F12.1)
  950 CONTINUE
C
C     SELECT THE BIGGER BUT THE CLOSEST SECTION TO THE CONT. SOLUTION ~
C
  955 DO 985 I=1,N
      WTI(I)=10.**9
      DO 980 J=1,NOAV
      IF (IPM(I)) 960,965,965
  960 IF (IAVDP(J)) 980,965,965
```

```
 965  IF (AVPM(J)-PM(I)) 980,970,970
 970  IF (AVWT(J)-WT1(I)) 975,980,980
 975  PM1(I)=AVPM(J)
      WT1(I)=AVWT(J)
      ID1(I)=J
 980  CONTINUE
 985  CONTINUE
C
C     CHECK CONSTRAINS FOR THE INITIAL VALUES
C
 990  CONTINUE
      DO 992 I=1,N
 992  X(I)=PM1(I)/X00
      CALL CONST(P,X,M,N,X00)
      DO 1035 I=1,M
      IF (P(I)) 995,1035,1035
 995  DO 1030 J=1,N
      K=ID1(J)
1000  K=K+1
      IF (NOAV-K) 1005,1015,1015
1005  WRITE(NWRIT,1010)
1010  FORMAT(1H1/10X,'THERE IS NO AVAILABLE SECTIONS')
      GO TO 60
1015  IF (IPM(J)) 1020,1025,1025
1020  IF (IAVDP(K)) 1000,1025,1025
1025  ID1(J)=K
      PM1(J)=AVPM(K)
      WT1(J)=AVWT(K)
1030  CONTINUE
      GO TO 990
1035  CONTINUE
C
C     CALCULATE THE TOTAL WEIGHT FOR THE INITIAL VALUE
C
      TWT1=0.
      DO 1040 J=1,N
1040  TWT1=TWT1+WT1(J)*FL(J)
C
C     DECREASE THE WEIGHT AS MUCH AS POSSIBLE & SELECT THE ECONOMICAL
C     SECTIONS
C
1045  IND=0
      DO 1050 J=1,N
```

```
      PM2(J)=PM1(J)
      WT2(J)=WT1(J)
      ID2(J)=ID1(J)
 1050 CONTINUE
      DO 1105 L=1,N
      K=ID2(L)
 1055 K=K-1
      IF (K) 1105,1105,1060
 1060 IF (AVWT(K)-WT2(L)) 1065,1105,1105
 1065 IF (IPM(L)) 1070,1075,1075
 1070 IF (IAVDP(K)) 1055,1075,1075
 1075 PM2(L)=AVPM(K)
      WT2(L)=AVWT(K)
      ID2(L)=K
C
C     CHECK CONSTRAINS
C
      DO 1076 I=1,N
 1076 X(I)=PM2(I)/XOO
      CALL CONST(P,X,M,N,XOO)
      DO 1078 I=1,M
      IF (P(I)) 1100,1100,1078
 1078 CONTINUE
 1080 CONTINUE
C
C     CALCULATE THE TOTAL WEIGHT FOR THE TRIAL VALUES
C
      TWT2=0.
      DO 1085 J=1,N
 1085 TWT2=TWT2+WT2(J)*FL(J)
      IF (TWT2-TWT1) 1090,1100,1100
 1090 IND=1
      PM11=PM1(L)
      WT11=WT1(L)
      ID11=ID1(L)
      DO 1095 J=1,N
      PM1(J)=PM2(J)
      WT1(J)=WT2(J)
      ID1(J)=ID2(J)
 1095 CONTINUE
      TWT1=TWT2
      PM2(L)=PM11
      WT2(L)=WT11
```

```
      IP2(L)=ID1(I)
      GO TO 1105
 1100 PM2(L)=PM1(I)
      WT2(L)=WT1(I)
      ID2(L)=ID1(I)
 1105 CONTINUE
      IF (IND) 1110,1110,1045
 1110 CONTINUE
      DO 1112 I=1,N
 1112 X(I)=PM1(I)/XOO
      CALL OBJECT (F,G,X,FL,WT,N,XOO,WOO)
      OBFN1=-F
      WRITE (NWRIT,1115)
 1115 FORMAT(1H1//17X,'CONTINUOUS SOLUTION          AVAILABLE SOLUTION'//)
      DO 1155 J=1,N
      K=ID1(J)
      K=IAVDP(K)
      IF(IPM(J)) 1145,1120,1120
 1120 IF (K) 1125,1135,1135
 1125 K=-K
      WRITE(NWRIT,1130)J,PM(J),PM1(J),K,WT1(J)
 1130 FORMAT(10X,'MP',I2,'(FT.KIPS)',F12.3,F12.3,'(',I4,'- B',F5.1,
     1'...SELECTED FROM AVAILABLE WF AND B SHAPES AS A BEAM MEMBER )')
      GO TO 1155
 1135 WRITE(NWRIT,1140)J,PM(J),PM1(J),K,WT1(J)
 1140 FORMAT(10X,'MP',I2,'(FT.KIPS)',F12.3,F12.3,'(',I4,' WF',F5.1,
     1'...SELECTED FROM AVAILABLE WF AND B SHAPES AS A BEAM MEMBER )')
      GO TO 1155
 1145 WRITE(NWRIT,1150)J,PM(J),PM1(J),K,WT1(J)
 1150 FORMAT(10X,'MP',I2,'(FT.KIPS)',F12.3,F12.3,'(',I4,' WF',F5.1,
     1'...SELECTED FROM AVAILABLE WF SHAPES AS A COLUMN MEMBER )')
 1155 CONTINUE
      TWT=TWT/1000.
      TWT1=TWT1/1000.
      WRITE(NWRIT,1160)OBFN,OBFN1,TWT,TWT1
 1160 FORMAT(/////10X,'OBJ. FUNCTION',2F12.3////10X,
     1'TOTAL WEIGHT ',2F12.3/16X,'(KIPS)')
      GO TO 60
 9999 STOP
      END
```

```
      SUBROUTINE INVERT(A,N)
      DIMENSION A(40,40)
      NWRIT=3
      N1=N-1
 1100 IND=1
C     DECOMPOSING A MATRIX INTO (UT)*(U)
      DO 245 I=1,N
      DO 240 J=1,N
      SUM=A(I,J)
      GO TO (210,200),IND
  200 I1=I-1
      DO 205 K=1,I1
  205 SUM=SUM-A(K,I)*A(K,J)
  210 IF (J-I) 215,220,215
  215 A(I,J)=SUM*TEMP
      GO TO 240
  220 IF (SUM) 225,225,235
  225 WRITE(NWRIT,230)
  230 FORMAT(/5X,'DIAGONAL ELEMENT OF (U*) LESS THAN ZERO'/)
      GO TO 9999
  235 TEMP=1./SQRT(SUM)
      A(I,J)=TEMP
  240 CONTINUE
      IND=2
  245 CONTINUE
C     INVERT MATRIX V OF U
      DO 255 I=1,N1
      I1=I+1
      DO 255 J=I1,N
      SUM=0.
      J1=J-1
      DO 250 K=I,J1
  250 SUM=SUM-A(K,I)*A(K,J)
  255 A(J,I)=SUM*A(J,J)
      DO 265 I=1,N
      DO 265 J=I,N
      SUM=0.
      DO 260 K=J,N
  260 SUM=SUM+A(K,I)*A(K,J)
      A(J,I)=SUM
      A(I,J)=A(J,I)
  265 CONTINUE
      RETURN
```

```
0042    0000 STOP
0043         END
```

```
0002          SUBROUTINE CONST( ,X,P, ,X  )
        C
        C     NONLINEAR INEQUALITIES
        C     TW -STORY   ONE -BAY FRAME
        C
0002          DIMENSION P(N),X(N)
0003          FL=0.
0004          FL2=FL*FL
        C
        C     MU-MP RELATION IS APPROXIMATED BY MU=A+B*MP+C*MP2+D*MP3
        C
0005          A=0.762452E 02
0006          B=0.169757E 01
0007          C=-0.134709E-02
0008          D=0.114787E-07
0009          XU1=A+B*X(1)*X00+C*(X(1)*X00)**2+D*(X(1)*X00)**3
0010          XU2=A+B*X(2)*X00+C*(X(2)*X00)**2+D*(X(2)*X00)**3
        C
        C     THE CONSTRAINTS WHICH GOVERN THE LINEAR PROGRAMMING SOLUTION
        C
0011          P(1)=2.*XU1+X(1)*X00+X(3)*X00-0.95*FL2
0012          U3=SQRT((XU2+X(2)*X00)/2.55)
0013          P(2)=2.*FL*XL2+(2.*FL-U3)*X(2)*X00+U3*X(3)*X00+5.*U3*X(4)*X00
             1-(372.54+2.55*FL2)*U3
0014          U1=SQRT((XU1+X(1)*X00)/1.7)
0015          U3=SQRT((XU2+X(2)*X00)/2.55)
0016          P(3)=2.*FL*U3*XU1+(2.*FL*U3-U1*U3)*X(1)*X00+2.*FL*U1*XU2
             1+(2.*FL*U1-U1*U3)*X(2)*X00+4.*U1*U3*X(3)*X00+5.*U1*U3*X(4)*X00
             2-(464.74+4.25*FL2)*U1*U3
0017          U1=SQRT((XU1+X(1)*X00)/1.7)
0018          U3=SQRT((XU2+X(2)*X00)/2.55)
0019          U4=SQRT((XU2+X(2)*X00+X(4)*X00)/2.55)
0020          P(4)=2.*FL*U3*U4*XU1+(2.*FL*U3*U4-U1*U3*U4)*X(1)*X00
             1+2.*FL*(U1*U4+U1*U3)*XU2+2.*FL*U1*U4*X(2)*X00
             2+2.*(U1*U3*U4+FL*U1*U3)*X(3)*X00+(3.*U1*U3*U4+2.*FL*U1*U3)*
             3X(4)*X00-(464.74+6.80*FL2)*U1*U3*U4
        C
        C     CONSTRAINTS CORRESPONDING TO ELEMENTARY MECHANISMS
        C
0021          P(5)=2.*XU1+2.*X(1)*X00-0.95*FL2
0022          P(6)=2.*XU2+2.*X(2)*X00-1.275*FL2
0023          P(7)=2.*XU2+X(2)*X00+X(3)*X00+X(4)*X00-1.275*FL2
0024          P(8)=6.*X(3)*X00-91.8
```

```
0025        F(1?)=2.*X(1)*X?? +?. X(3)*X??-??.
0026        P(1?)=4.*X(?)*X??+3. X(3)*X??-??.
0027        P(??)=4.*X(4)*X??-3??.?4
          C
          C     ADDITIONAL CONSTRAINTS
          C
0028        U1=SQRT((XU1+X(1)*X00)/1.7)
0029        U2=SQRT((XU2+X(2)*X00)/2.55)
0030        P(12)=2.*FL*U2*XU1+2.*FL*U3*X(1)*X00+2.*FL*U1*XU2
           1+(2.*FL*U1-U1*U2)*X(2)*X00+3.*U1*U3*X(3)*X00+5.*U1*U3*X(4)*X00
           2-(464.74+4.25*FL2)*U1*U3
0031        U1=SQRT((XU1+X(1)*X00)/1.7)
0032        U3=SQRT((XU2+X(2)*X00)/2.55)
0033        P(13)=2.*FL*U3*XU1+(2.*FL*U2-U1*U3)*X(1)*X00+2.*FL*U1*XU2
           1+2.*FL*U1*X(2)*X00+3.*U1*U3*X(3)*X00+4.*U1*U3*X(4)*X00
           2-(464.74+4.25*FL2)*U1*U3
0034        U1=SQRT((XU1+X(1)*X00)/1.7)
0035        U3=SQRT((XU2+X(2)*X00)/2.55)
0036        P(14)=2.*FL*U3*XU1+2.*FL*U3*X(1)*X00+2.*FL*U1*XU2
           1+2.*FL*U1*X(2)*X00+2.*U1*U3*X(3)*X00+4.*U1*U3*X(4)*X00
           2-(464.74+4.25*FL2)*U1*U3
0037        U3=SQRT((XU2+X(2)*X00)/2.55)
0038        U4=SQRT((XU2+X(2)*X00+X(4)*X00)/2.55)
0039        P(15)=2.*FL*(U3+U4)*XU2+2.*FL*U4*X(2)*X00
           1+(U3*U4+2.*FL*U3)*X(3)*X00+(4.*U3*U4+2.*FL*U3)*X(4)*X00
           2-(372.94+5.10*FL2)*U3*U4
0040        U1=SQRT((XU1+X(1)*X00)/1.7)
0041        U2=SQRT((XU1+X(1)*X00)/1.7)
0042        U3=SQRT((XU2+X(2)*X00)/2.55)
0043        P(16)=2.*FL*(U1*U3+U2*U3)*XU1+2.*FL*(U1*U3+U2*U3)*X(1)*X00
           1+2.*FL*U1*U2*XU2+(2.*FL*(U1+U2)-U1*U2*U3)*X(2)*X00
           2+2.*U1*U2*U3*X(3)*X00+5.*U1*U2*U3*X(4)*X00
           3-(464.74+5.95*FL2)*U1*U2*U3
0044        U1=SQRT((XU1+X(1)*X00)/1.7)
0045        U2=SQRT((XU1+X(1)*X00)/1.7)
0046        U3=SQRT((XU2+X(2)*X00)/2.55)
0047        P(17)=2.*FL*(U1*U3+U2*U3)*XU1+2.*FL*U2*U3*X(1)*X00+2.*FL*U1*U2*XU2
           1+(2.*FL*U1*U2-U1*U2*U3)*X(2)*X00+2.*(U1*U2*U3+FL*U1*U3)*X(3)*X00
           2+5.*U1*U2*U3*X(4)*X00
           3-(464.74+5.95*FL2)*U1*U2*U3
0048        U1=SQRT((XU1+X(1)*X00)/1.7)
0049        U2=SQRT((XU1+X(1)*X00)/1.7)
0050        U3=SQRT((XU2+X(2)*X00)/2.55)
```

```
0051       P(19)=2.*FL*(U1*U3+U2*U2)*XU2+2.*FL*(U1*U2+U2*U3)*X(1)*X00
          1+2.*FL*(U1*X(2)*X00+(U1*U2*U3+2.*FL*U1*U2)*X(2)*X00
          2+4.*U1*U2*U3*X(4)*X00
          3-(444.74+5.55*FL2)-U1*U2*U3
0052       U1=SQRT((XU1+X(1)*X00)/1.7)
0053       U2=SQRT((XU1+X(3)*X00)/1.7)
0054       U3=SQRT((XU2+X(2)*X00)/2.55)
0055       P(19)=2.*FL*(U1*U2+U2*U3)*XU1+2.*FL*U2*U3*X(1)*X00+2.*FL*U1*U2*XU2
          1+2.*FL*U1*U2*X(2)*X00+(U1*U2*U3+2.*FL*U1*U3)*X(3)*X00
          2+4.*U1*U2*U3*X(4)*X00
          2-(444.74+5.55*FL2)*U1*U2*U3
0056       U3=SQRT((XU2+X(2)*X00)/2.55)
0057       U4=SQRT((XU2+X(3)*X00+X(4)*X00)/2.55)
0058       P(20)=2.*FL*(U3+U4)*XU2+2.*FL*U4*X(2)*X00
          1+2.*(U3*U4+FL*U3)*X(3)*X00+(3.*U3*U4+2.*FL*U3)*X(4)*X00
          2-(372.54+5.10*FL2)*U3*U4
0059       U3=SQRT((XU2+X(2)*X00)/2.55)
0060       U4=SQRT((XU2+X(2)*X00)/2.55)
0061       P(21)=2.*FL*(U3+U4)*XU2+(2.*FL*U3+2.*FL*U4-U3*U4)*X(2)*X00
          1+2.*U3*U4*X(3)*X00+4.*U3*U4*X(4)*X00
          2-(372.54+5.10*FL2)*U3*U4
0062       U3=SQRT((XU2+X(2)*X00)/2.55)
0063       U4=SQRT((XU2+X(3)*X00+X(4)*X00)/2.55)
0064       P(22)=2.*FL*(U3+U4)*XU2+2.*FL*U4*X(2)*X00
          1+(3.*U3*U4+2.*FL*U3)*X(3)*X00+(3.*U3*U4+2.*FL*U3)*X(4)*X00
          2-(444.74+5.10*FL2)*U3*U4
0065       U3=SQRT((XU2+X(2)*X00)/2.55)
0066       U4=SQRT((XU2+X(2)*X00)/2.55)
0067       P(23)=2.*FL*(U3+U4)*XU2+2.*FL*(U3+U4)*X(2)*X00+3.*U3*U4*X(3)*X00
          1+3.*U3*U4*X(4)*X00
          2-(444.74+5.10*FL2)*U3*U4
0068       U3=SQRT((XU2+X(2)*X00)/2.55)
0069       U4=SQRT((XU2+X(3)*X00+X(4)*X00)/2.55)
0070       P(24)=2.*U3*U4*X(1)*X00+2.*FL*(U3+U4)*XU2+2.*FL*U4*X(2)*X00
          1+(U3*U4+2.*FL*U3)*X(3)*X00+(2.*U3*U4+2.*FL*U3)*X(4)*X00
          2-(444.74+5.10*FL2)*U3*U4
0071       U3=SQRT((XU2+X(2)*X00)/2.55)
0072       U4=SQRT((XU2+X(2)*X00)/2.55)
0073       P(25)=2.*U3*U4*X(1)*X00+2.*FL*(U3+U4)*XU2+2.*FL*(U3+U4)*X(2)*X00
          1+U3*U4*X(3)*X00+2.*U3*U4*X(4)*X00
          2-(444.74+5.10*FL2)*U3*U4
0074       U1=SQRT((XU1+X(1)*X00)/1.7)
0075       U2=SQRT((XU1+X(1)*X00)/1.7)
```

```
0076        U3=SQRT((XU2+X(2)*XOO)/2.55)
0077        U4=SQRT((XU2+X(3)*XOO+X(4)*XOO)/2.55)
0078        P(26)=2.*FL*(U2*U3*U4+U1*U3*U4)*XU1
           1+2.*FL*(U2*U3*U4+U1*U3*U4)*X(1)*XOO
           2+2.*FL*(U1*U2*U4+U1*U2*U3)*XU2+2.*FL*U1*U2*U4*X(2)*XOO
           3+2.*FL*U1*U2*U3*X(3)*XOO+(3.*U1*U2*U3*U4+2.*FL*U1*U2*U3)*X(4)*XOO
           4-(464.74+8.50*FL2)*U1*U2*U3*U4
0079        U1=SQRT((XU1+X(1)*XOO)/1.7)
0080        U2=SQRT((XU1+X(1)*XOO)/1.7)
0081        U3=SQRT((XU2+X(2)*XOO)/2.55)
0082        U4=SQRT((XU2+X(3)*XOO+X(4)*XOO)/2.55)
0083        P(27)=2.*FL*(U2*U3*U4+U1*U3*U4)*XU1+2.*FL*U2*U3*U4*X(1)*XOO
           1+2.*FL*(U1*U2*U4+U1*U2*U3)*XU2+2.*FL*U1*U2*U4*X(2)*XOO
           2+2.*FL*(U1*U3*U4+U1*U2*U3)*X(3)*XOO
           3+(3.*U1*U2*U3*U4+2.*FL*U1*U2*U3)*X(4)*XOO
           4-(464.74+8.50*FL2)*U1*U2*U3*U4
0084        U1=SQRT((XU1+X(1)*XOO)/1.7)
0085        U2=SQRT((XU1+X(1)*XOO)/1.7)
0086        U3=SQRT((XU2+X(2)*XOO)/2.55)
0087        U4=SQRT((XU2+X(2)*XOO)/2.55)
0088        P(28)=2.*FL*(U2*U3*U4+U1*U3*U4)*XU1
           1+2.*FL*(U2*U3*U4+U1*U3*U4)*X(1)*XOO
           2+2.*FL*(U1*U2*U4+U1*U2*U3)*XU2
           3+2.*FL*(U1*U2*U4+U1*U2*U3)*X(2)*XOO+3.*U1*U2*U3*U4*X(4)*XOO
           4-(464.74+8.50*FL2)*U1*U2*U3*U4
0089        U1=SQRT((XU1+X(1)*XOO)/1.7)
0090        U2=SQRT((XU1+X(3)*XOO)/1.7)
0091        U3=SQRT((XU2+X(2)*XOO)/2.55)
0092        U4=SQRT((XU2+X(2)*XOO)/2.55)
0093        P(29)=2.*FL*(U2*U3*U4+U1*U3*U4)*XU1+2.*FL*U2*U3*U4*X(1)*XOO
           1+2.*FL*(U1*U2*U4+U1*U2*U3)*XU2
           2+2.*FL*(U1*U2*U3+U1*U2*U4)*X(2)*XOO
           3+2.*FL*U1*U3*U4*X(3)*XOO+3.*U1*U2*U3*U4*X(4)*XOO
           4-(464.74+8.50*FL2)*U1*U2*U3*U4
0094        U1=SQRT((XU1+X(1)*XOO)/1.7)
0095        U3=SQRT((XU2+X(2)*XOO)/2.55)
0096        U4=SQRT((XU2+X(3)*XOO+X(4)*XOO)/2.55)
0097        P(30)=2.*FL*U3*U4*XU1+2.*FL*U3*U4*X(1)*XOO
           1+2.*FL*(U1*U4+U1*U3)*XU2+2.*FL*U1*U4*X(2)*XOO
           2+(U1*U3*U4+2.*FL*U1*U3)*X(3)*XOO
           3+(3.*U1*U3*U4+2.*FL*U1*U3)*X(4)*XOO
           4-(464.74+6.80*FL2)*U1*U3*U4
0098        U1=SQRT((XU1+X(1)*XOO)/1.7)
```

```
0099      U2=SQRT((XU2+X(2)*X00)/2.[illegible])
0100      U4=SQRT((XU2+X(2)*X00)/2.[illegible])
0101      F(31)=2.*FL*(U3*U4*XU1+(2.*FL*U3*U4-U1*U3*U4)*X(1)*X00
         1+2.*FL*(U1*U3+U1*U4)*XU2+2.*FL*(U1*U3+U1*U4)*X(2)*X00
         2+2.*U1*U3*U4*X(3)-X00+2.*U1*U3*U4*X(4)*X00
         3-(444.74+6.80*FL**2)*U1*U3*U4
0102      U1=SQRT((XU1+X(1)*X00)/3.7)
0103      U3=SQRT((XU2+X(2)*X00)/2.55)
0104      U4=SQRT((XU2+X(2)*X00)/2.55)
0105      F(32)=2.*FL*U3*U4*XU1+2.*FL*U3*U4*X(1)*X00
         1+2.*FL*(U1*U4+U1*U3)*XU2+2.*FL*(U1*U3+U1*U4)*X(2)*X00
         2+U1*U3*U4*X(3)*X00+2.*U1*U3*U4*X(4)*X00
         3-(464.74+6.80*FL**2)*U1*U3*U4
0106      RETURN
0107      END
```

```
      [illegible]
      [illegible]
      A=1.[illegible]
      B=0.[illegible]
      E3=1.-[illegible]
      DO 10 I=1,N
      C(I)=-1./W[illegible]/((X(I)*Y[illegible])*[illegible])
      WT(I)=A*(X(I)*Y[illegible])**[illegible]
   10 CONTINUE
      W=0.
      DO 20 I=1,N
      W=W+WT(I)*FL(I)
   20 CONTINUE
      F=-W/W[illegible]
      RETURN
      END
```

F88-LEVEL LINKAGE EDITOR OPTIONS SPECIFIED MAP,LIST,LET
DEFAULT OPTION(S) USED - SIZE=(102400,14336)

MODULE MAP

CONTROL SECTION			ENTRY							
NAME	ORIGIN	LENGTH	NAME	LOCATION	NAME	LOCATION	NAME	LOCATION	NAME	LOCATION
MAIN	00	7250								
INVECT	7250	5D8								
CONST	7828	2734								
*PROJECT	9F60	7F4								
IHCEFXPI*	A758	141								
IHCEFXPF*	A8A0	193	FRXPI#	A758						
IHCFCOMH*	[illegible]	F41	FRXPR#	A8A0						
			IBCOM#	A5E8	FDIOCS#	A5F4	INTSWTCH	B4EE		
IHCCCVH2*	B520	65D	SEQDASD	B539						
IHCSSCRT*	BB80	14F								
IHCFCVTH*	BCD8	1190	SQRT	BB80						
			ADCON#	BCD8	FCVAOUTP	[illegible]	FCVLOUTP	[illegible]	FCVZOUTP	BF52
IHCSLOG *	CE68	104	FCVIOUTP	C300	FCVEOUTP	[illegible]	FCVCOUTP	[illegible]	INTSWCH	C003
			ALOG10	CE68	ALOG	CE80				
IHCEFNTH*	D020	543								
			ARITH#	D020	ADJSWTCH	D7BC				
IHCSEXP *	D568	1A2								
			EXP	D568						
IHCFFIOS*	D700	5BC								
			FIOCS#	D700	FIOCSBEP	D704				
IHCFIOS2*	E5C0	5DC								
IHCERRM *	EAF0	5D4								
			ERRMON	EAF0	IHCERRE	EB0A				
IHCUOPT *	F0C8	2C0								
IHCETRCH*	F3C8	28F								
			IHCTRCH	F3C8	ERRTRA	F3D0				
IHCUATBL*	F658	368								

ENTRY ADDRESS 00
TOTAL LENGTH F7C0

****MAIN DOES NOT EXIST BUT HAS BEEN ADDED TO DATA SET

A. 4-9 Output Example for the Nonlinear Optimization of the Composite Frame in Sec. 6-6

THE NUMBER OF CONSTRAINT FUNCTION M= 32

THE NUMBER OF DESIGN VARIABLES N= 4

THE TOLERANCE EPS= 0.99999997E-07 (WHICH CONTROLS THE NEARNESS OF THE POINT XMAX TO THE TRUE OPTIMUM POINT)

THE TOLERANCE DELTA= 0.99999994E-06 (WHICH CONTROLS THE NEARNESS OF THE POINT OV TO THE INTERSECTION G)

THE REFERANCE VALUE XOO= 1000. (USED FOR DEFINING THE NON-DIMENSIONAL DESIGN VARIABLES)

THE REFERANCE VALUE WOO= 1000. (USED FOR DEFINING THE NON-DIMENSIONAL OBJECTIVE FUNCTION)

	THE MEMBER LENGTH L	MEMBER CODE (WHERE NEGATIVE AND POSITIVE CODES SHOW COLUMN AND BEAM MEMBERS RESPECTIVELY)
1	60.00000	1
2	60.00000	1
3	36.00000	-1
4	45.00000	-1

THE INITIAL DESIGN VARIABLES X*, NONDIMENSIONALIZED VARIABLES X, UPPER BOUNDS BU AND LOWER BOUNDS BL ARE AS FOLLOWS

	X*	X	BU	BL
1	300.00000	0.300000	1.000000	0.0
2	500.00000	0.500000	1.000000	0.0
3	300.00000	0.300000	1.000000	0.0
4	200.00000	0.200000	1.000000	0.0

```
W/WOO=        11.31203
X(   1)=  0.2904256E 00
X(   2)=  0.4920152E 00
X(   3)=  0.1889416E 00
X(   4)=  0.1889416E 00
P (   1 )=  0.8251382E 03
P (   2 )=  0.3512560E 05
P (   3 )=  0.1263759E 07
P (   4 )=  0.3756046E 08
P (   5 )=  0.9266223E 03
P (   6 )=  0.1587503E 04
P (   7 )=  0.1473271E 04
P (   8 )=  0.1041849E 04
P (   9 )=  0.1244817E 04
P (  10 )=  0.1636727E 04
P (  11 )=  0.7607087E 03
P (  12 )=  0.1316179E 07
P (  13 )=  0.1322712E 07
P (  14 )=  0.1375133E 07
P (  15 )=  0.1497576E 07
P (  16 )=  0.3576670E 08
P (  17 )=  0.3203006E 08
P (  18 )=  0.2708170E 08
P (  19 )=  0.3326365E 08
P (  20 )=  0.1497576E 07
P (  21 )=  0.1482763E 07
P (  22 )=  0.1547449E 07
P (  23 )=  0.1697388E 07
P (  24 )=  0.1651654E 07
P (  25 )=  0.1806234E 07
P (  26 )=  0.1005907E 10
P (  27 )=  0.9097864E 09
P (  28 )=  0.1090913E 10
P (  29 )=  0.9881203E 09
P (  30 )=  0.3872264E 08
P (  31 )=  0.4104051E 08
P (  32 )=  0.4225446E 08

W/WOO=        11.07302
X(   1)=  0.2803934E 00
X(   2)=  0.4841115E 00
X(   3)=  0.1778359E 00
X(   4)=  0.1778359E 00

W/WOO=        10.83056
X(   1)=  0.2714093E 00
X(   2)=  0.4762956E 00
X(   3)=  0.1666787E 00
X(   4)=  0.1666787E 00

W/WOO=        10.58444
X(   1)=  0.2619799E 00
X(   2)=  0.4685746E 00
X(   3)=  0.1554654E 00
X(   4)=  0.1554654E 00

W/WOO=         8.12768
X(   1)=  0.1818498E 00
X(   2)=  0.4053903E 00
X(   3)=  0.5071073E-01
X(   4)=  0.5071073E-01
```

X(1)= 0.1835773E 00
X(2)= 0.4112593E 00
X(3)= 0.6215667E-01
X(4)= 0.6215667E-01

C = 1

PHI-ACTIVE (1)=0.		PHI (1)=	0.2696733E 03
PHI-ACTIVE (2)=0.		PHI (2)=	0.1186906E 05
PHI-ACTIVE (3)=0.		PHI (3)=	0.3222650E 06
PHI-ACTIVE (4)=0.		PHI (4)=	0.1105419E 08
PHI-ACTIVE (5)=0.		PHI (5)=	0.3970898E 03
PHI-ACTIVE (6)=0.		PHI (6)=	0.1171893E 04
PHI-ACTIVE (7)=0.		PHI (7)=	0.8849468E 03
PHI-ACTIVE (8)=0.		PHI (8)=	0.2811399E 03
PHI-ACTIVE (9)=0.		PHI (9)=	0.5359722E 03
PHI-ACTIVE (10)=0.		PHI (10)=	0.8529617E 03
PHI-ACTIVE (11)=1.		PHI (11)=	0.0
PHI-ACTIVE (12)=0.		PHI (12)=	0.3724990E 06
PHI-ACTIVE (13)=0.		PHI (13)=	0.4353960E 06
PHI-ACTIVE (14)=0.		PHI (14)=	0.4855300E 06
PHI-ACTIVE (15)=0.		PHI (15)=	0.5685880E 06
PHI-ACTIVE (16)=0.		PHI (16)=	0.9026512E 07
PHI-ACTIVE (17)=0.		PHI (17)=	0.6561728E 07
PHI-ACTIVE (18)=0.		PHI (18)=	0.1111803E 08
PHI-ACTIVE (19)=0.		PHI (19)=	0.8409760E 07
PHI-ACTIVE (20)=0.		PHI (20)=	0.[illegible]5880E 06
PHI-ACTIVE (21)=0.		PHI (21)=	0.6932670E 06
PHI-ACTIVE (22)=0.		PHI (22)=	0.5568930E 06
PHI-ACTIVE (23)=0.		PHI (23)=	0.8385510E 06
PHI-ACTIVE (24)=0.		PHI (24)=	0.6574310E 06
PHI-ACTIVE (25)=0.		PHI (25)=	0.9544440E 06
PHI-ACTIVE (26)=0.		PHI (26)=	0.2611343E 09
PHI-ACTIVE (27)=0.		PHI (27)=	0.2045750E 09
PHI-ACTIVE (28)=0.		PHI (28)=	0.3682143E 09
PHI-ACTIVE (29)=0.		PHI (29)=	0.2951941E 09
PHI-ACTIVE (30)=0.		PHI (30)=	0.1198355E 09
PHI-ACTIVE (31)=0.		PHI (31)=	0.1637722E 09
PHI-ACTIVE (32)=0.		PHI (32)=	0.1744851E 09

```
W/WOO=          8.11016
WOLD-W=  0.9999997E 20
ABS(PQG)=   0.2705595E-03        B(IDROP)= -0.1033393E-03
BET =   0.2705595E-03
ITERATION PROCEDURE IS REQUIRED

W/WOO=          7.85464
WOLD-W=  0.2555195E 03
ABS(PQG)=  0.2891729E-03        B(IDROP)= -0.1033394E-03
BET =  0.2891729E-03
ITERATION PROCEDURE IS REQUIRED

WQ (   1 )=   0.0

W/WOO=          7.59402
WOLD-W=  0.2706211E 03
ABS(PQG)=   0.3198532E-03        B(IDROP)= -0.1033393E-03
BET = -0.3198532E-03
ITERATION PROCEDURE IS REQUIRED
```

```
UNABLE TO ADD NEXT CONSTRAINT

UNABLE TO SATISFY THE TOLERANCE DELTA. HENCE RESET DELTA TO     0.4577637E-04
X(   1)=   0.1608503E 00
X(   2)=   0.3857150E 00
X(   3)=   0.1530001E-01
X(   4)=   0.6215669E-01


Q =    2


PHI-ACTIVE (   1 )=0.             PHI (   1 )=  0.9978052E 02
PHI-ACTIVE (   2 )=0.             PHI (   2 )=  0.9403297E 04
PHI-ACTIVE (   3 )=0.             PHI (   3 )=  0.1698250E 06
PHI-ACTIVE (   4 )=0.             PHI (   4 )=  0.7010416E 07
PHI-ACTIVE (   5 )=0.             PHI (   5 )=  0.2453311E 03
PHI-ACTIVE (   6 )=0.             PHI (   6 )=  0.1060486E 04
PHI-ACTIVE (   7 )=0.             PHI (   7 )=  0.7482273E 03
PHI-ACTIVE (   8 )=1.             PHI (   8 )=  0.4577637E-04
PHI-ACTIVE (   9 )=0.             PHI (   9 )=  0.2911003E 03
PHI-ACTIVE (  10 )=0.             PHI (  10 )=  0.5975010E 03
PHI-ACTIVE (  11 )=1.             PHI (  11 )=  0.0
PHI-ACTIVE (  12 )=0.             PHI (  12 )=  0.2220300E 06
PHI-ACTIVE (  13 )=0.             PHI (  13 )=  0.2818250E 06
PHI-ACTIVE (  14 )=0.             PHI (  14 )=  0.3340300E 06
PHI-ACTIVE (  15 )=0.             PHI (  15 )=  0.4327860E 06
PHI-ACTIVE (  16 )=0.             PHI (  16 )=  0.5144368E 07
PHI-ACTIVE (  17 )=0.             PHI (  17 )=  0.2907040E 07
PHI-ACTIVE (  18 )=0.             PHI (  18 )=  0.7075072E 07
PHI-ACTIVE (  19 )=0.             PHI (  19 )=  0.4536016E 07
PHI-ACTIVE (  20 )=0.             PHI (  20 )=  0.4156060E 06
PHI-ACTIVE (  21 )=0.             PHI (  21 )=  0.5600350E 06
PHI-ACTIVE (  22 )=0.             PHI (  22 )=  0.3875600E 06
PHI-ACTIVE (  23 )=0.             PHI (  23 )=  0.6687280E 06
PHI-ACTIVE (  24 )=0.             PHI (  24 )=  0.4942870E 06
PHI-ACTIVE (  25 )=0.             PHI (  25 )=  0.7947560E 06
PHI-ACTIVE (  26 )=0.             PHI (  26 )=  0.1599091E 09
PHI-ACTIVE (  27 )=0.             PHI (  27 )=  0.1095620E 09
PHI-ACTIVE (  28 )=0.             PHI (  28 )=  0.2515517E 09
PHI-ACTIVE (  29 )=0.             PHI (  29 )=  0.1824156E 09
PHI-ACTIVE (  30 )=0.             PHI (  30 )=  0.7930304E 07
PHI-ACTIVE (  31 )=0.             PHI (  31 )=  0.1191686E 08
PHI-ACTIVE (  32 )=0.             PHI (  32 )=  0.1300310E 08
```

```
W/WOO=          7.29897

WOLD-W=  0.2850509E 03

ABS(POG)=   0.1825276E-03          B(IDROP)= -0.1033393E-03

BET =  0.1825276E-03

ITERATION PROCEDURE IS REQUIRED

X(    1)=  0.1446977E 00
X(    2)=  0.3779212E 00
X(    3)=  0.1530001E-01
X(    4)=  0.6215667E-01

Q =    2

W/WOO=          7.07695

WOLD-W=  0.2221172E 03

ABS(POG)=   0.1878026E-03          B(IDROP)= -0.1033393E-03

BET =  0.1878026E-03

ITERATION PROCEDURE IS REQUIRED

X(    1)=  0.1283970E 00     CORR (   1 )=  0.0
X(    2)=  0.3663327E 00     CORR (   2 )=  0.0
X(    3)=  0.1530000E-01     CORR (   3 )= -0.2311786E-07
X(    4)=  0.6215666E-01     CORR (   4 )=  0.0

INTERPOLATION PROCEDURE IS REQUIRED
```

```
X(    1)=   0.1376293E 00
X(    2)=   0.3728361E 00
X(    3)=   0.1530001E-01
X(    4)=   0.6215667E-01

C =   2

PHI-ACTIVE (    1  )=1.          PHI  (   1  )=   0.0
PHI-ACTIVE (    2  )=0.          PHI  (   2  )=   0.8265117E 04
PHI-ACTIVE (    3  )=0.          PHI  (   3  )=   0.1119660E 06
PHI-ACTIVE (    4  )=0.          PHI  (   4  )=   0.5207136E 07
PHI-ACTIVE (    5  )=0.          PHI  (   5  )=   0.1223296E 03
PHI-ACTIVE (    6  )=0.          PHI  (   6  )=   0.9733586E 03
PHI-ACTIVE (    7  )=0.          PHI  (   7  )=   0.6779187E 03
PHI-ACTIVE (    8  )=1.          PHI  (   8  )=   0.4577637E-04
PHI-ACTIVE (    9  )=0.          PHI  (   9  )=   0.2446585E 03
PHI-ACTIVE (   10  )=0.          PHI  (  10  )=   0.5046169E 03
PHI-ACTIVE (   11  )=1.          PHI  (  11  )=   0.0
PHI-ACTIVE (   12  )=0.          PHI  (  12  )=   0.1521660E 06
PHI-ACTIVE (   13  )=0.          PHI  (  13  )=   0.2091960E 06
PHI-ACTIVE (   14  )=0.          PHI  (  14  )=   0.2494950E 06
PHI-ACTIVE (   15  )=0.          PHI  (  15  )=   0.3733060E 06
PHI-ACTIVE (   16  )=0.          PHI  (  16  )=   0.3003696E 07
PHI-ACTIVE (   17  )=0.          PHI  (  17  )=   0.1444592E 07
PHI-ACTIVE (   18  )=0.          PHI  (  18  )=   0.4576032E 07
PHI-ACTIVE (   19  )=0.          PHI  (  19  )=   0.2782720E 07
PHI-ACTIVE (   20  )=0.          PHI  (  20  )=   0.3617560E 06
PHI-ACTIVE (   21  )=0.          PHI  (  21  )=   0.4921650E 06
PHI-ACTIVE (   22  )=0.          PHI  (  22  )=   0.3347350E 06
PHI-ACTIVE (   23  )=0.          PHI  (  23  )=   0.5895730E 06
PHI-ACTIVE (   24  )=0.          PHI  (  24  )=   0.4211500E 06
PHI-ACTIVE (   25  )=0.          PHI  (  25  )=   0.6913150E 06
PHI-ACTIVE (   26  )=0.          PHI  (  26  )=   0.1048440E 09
PHI-ACTIVE (   27  )=0.          PHI  (  27  )=   0.6997171E 08
PHI-ACTIVE (   28  )=0.          PHI  (  28  )=   0.1744522E 09
PHI-ACTIVE (   29  )=0.          PHI  (  29  )=   0.1257961E 09
PHI-ACTIVE (   30  )=0.          PHI  (  30  )=   0.5905136E 07
PHI-ACTIVE (   31  )=0.          PHI  (  31  )=   0.9288432E 07
PHI-ACTIVE (   32  )=0.          PHI  (  32  )=   0.1011026E 08
```

```
W/WOO=          6.27568
WELD-W=  0.9837391E 02
ABS(PQG)=  0.1093356E-03        B(IPROP)= -0.7790662E-04
BET =  0.1093356E-03
ITERATION PROCEDURE IS REQUIRED
W/WOO=          6.71[illegible]
WELD-W=  0.1351836E 03
ABS(PQG)=  0.1138349E-03        B(IPROP)= -0.7790668E-04
BET =  0.1138349E-03
ITERATION PROCEDURE IS REQUIRED
W/WOO=          6.57271
WELD-W=  0.1381116E 03
ABS(PQG)=  0.1163654E-03        B(IPROP)= -0.7790668E-04
BET =  0.1163654E-03
ITERATION PROCEDURE IS REQUIRED
W/WOO=          6.28669
WELD-W=  0.1447500E 03
ABS(PQG)=  0.1221660E-03        B(IPROP)= -0.7790668E-04
BET =  0.1221660E-03
ITERATION PROCEDURE IS REQUIRED
```

```
X(    1)=  0.1374233E 01
X(    2)=  0.2702319E 00
X(    3)=  0.1530001E-01
X(    4)=  0.6215667E-01

C =    4

PHI-ACTIVE  (   1  )=1.          PHI  (   1  )=  0.0
PHI-ACTIVE  (   2  )=0.          PHI  (   2  )=  0.1523176E 04
PHI-ACTIVE  (   3  )=0.          PHI  (   3  )=  0.5878000E 04
PHI-ACTIVE  (   4  )=0.          PHI  (   4  )=  0.8251040E 06
PHI-ACTIVE  (   5  )=0.          PHI  (   5  )=  0.1222293E 03
PHI-ACTIVE  (   6  )=0.          PHI  (   6  )=  0.4390283E 03
PHI-ACTIVE  (   7  )=0.          PHI  (   7  )=  0.2461528E 03
PHI-ACTIVE  (   8  )=1.          PHI  (   8  )=  0.4577637E-04
PHI-ACTIVE  (   9  )=0.          PHI  (   9  )=  0.2446583E 03
PHI-ACTIVE  (  10  )=0.          PHI  (  10  )=  0.5046167E 03
PHI-ACTIVE  (  11  )=1.          PHI  (  11  )=  0.0
PHI-ACTIVE  (  12  )=0.          PHI  (  12  )=  0.4073400E 05
PHI-ACTIVE  (  13  )=0.          PHI  (  13  )=  0.6083400E 05
PHI-ACTIVE  (  14  )=0.          PHI  (  14  )=  0.9568900E 05
PHI-ACTIVE  (  15  )=0.          PHI  (  15  )=  0.9396800E 05
PHI-ACTIVE  (  16  )=0.          PHI  (  16  )=  0.1129824E 07
PHI-ACTIVE  (  17  )=1.          PHI  (  17  )=  0.0
PHI-ACTIVE  (  18  )=0.          PHI  (  18  )=  0.2017616E 07
PHI-ACTIVE  (  19  )=0.          PHI  (  19  )=  0.7555520E 06
PHI-ACTIVE  (  20  )=0.          PHI  (  20  )=  0.8128600E 05
PHI-ACTIVE  (  21  )=0.          PHI  (  21  )=  0.1378270E 06
PHI-ACTIVE  (  22  )=0.          PHI  (  22  )=  0.6058200E 05
PHI-ACTIVE  (  23  )=0.          PHI  (  23  )=  0.1787890E 06
PHI-ACTIVE  (  24  )=0.          PHI  (  24  )=  0.1267960E 06
PHI-ACTIVE  (  25  )=0.          PHI  (  25  )=  0.2548980E 06
PHI-ACTIVE  (  26  )=0.          PHI  (  26  )=  0.2920806E 08
PHI-ACTIVE  (  27  )=0.          PHI  (  27  )=  0.1010355E 08
PHI-ACTIVE  (  28  )=0.          PHI  (  28  )=  0.6206003E 08
PHI-ACTIVE  (  29  )=0.          PHI  (  29  )=  0.3585690E 08
PHI-ACTIVE  (  30  )=0.          PHI  (  30  )=  0.1359920E 07
PHI-ACTIVE  (  31  )=0.          PHI  (  31  )=  0.2711744E 07
PHI-ACTIVE  (  32  )=0.          PHI  (  32  )=  0.3326496E 07
```

```
W/WOC=          6.26785

WOLD-W=  0.16824525 02

ABS(PQG)=  0.14051465E-08          B(IPROP)=  0.2769475E-04

BET =  0.2769475E-04

IPROP=   1             Q=   3


W/WOC=          6.26785

WOLD-W=  0.0

ABS(PQG)=  0.55398355E-04          B(IPROP)= -0.45561205E-04

BET =  0.55368835E-04

ITERATION PROCEDURE IS REQUIRED

X(   1)=  0.14534525 00     CORR (   1 )=  0.41058805E-05
X(   2)=  0.26187845 00     CORR (   2 )=  0.16561905E-06
X(   3)=  0.15300005E-01    CORR (   3 )=  0.77062055E-08
X(   4)=  0.62156695E-01    CORR (   4 )=  0.28421711E-12

INTERPOLATION PROCEDURE IS REQUIRED
```

X(1)=	0.1421309E 00
X(2)=	0.2594205E 00
X(3)=	0.1530001E-01
X(4)=	0.6213668E-01

Q = 4

PHI-ACTIVE (1)=0.		PHI (1)=	0.1936548E 02
PHI-ACTIVE (2)=0.		PHI (2)=	0.8350039E 03
PHI-ACTIVE (3)=1.		PHI (3)=	0.0
PHI-ACTIVE (4)=0.		PHI (4)=	0.5244960E 06
PHI-ACTIVE (5)=0.		PHI (5)=	0.1461965E 03
PHI-ACTIVE (6)=0.		PHI (6)=	0.3818750E 03
PHI-ACTIVE (7)=0.		PHI (7)=	0.1999109E 03
PHI-ACTIVE (8)=1.		PHI (8)=	0.4577637E-04
PHI-ACTIVE (9)=0.		PHI (9)=	0.2536617E 03
PHI-ACTIVE (10)=0.		PHI (10)=	0.5226233E 03
PHI-ACTIVE (11)=1.		PHI (11)=	0.0
PHI-ACTIVE (12)=0.		PHI (12)=	0.3595500E 05
PHI-ACTIVE (13)=0.		PHI (13)=	0.5159500E 05
PHI-ACTIVE (14)=0.		PHI (14)=	0.8754000E 05
PHI-ACTIVE (15)=0.		PHI (15)=	0.6894300E 05
PHI-ACTIVE (16)=0.		PHI (16)=	0.1166448E 07
PHI-ACTIVE (17)=1.		PHI (17)=	0.0
PHI-ACTIVE (18)=0.		PHI (18)=	0.201[illegible]E 07
PHI-ACTIVE (19)=0.		PHI (19)=	0.7173760E 06
PHI-ACTIVE (20)=0.		PHI (20)=	0.5667700E 05
PHI-ACTIVE (21)=0.		PHI (21)=	0.1067140E 06
PHI-ACTIVE (22)=0.		PHI (22)=	0.3665000E 05
PHI-ACTIVE (23)=0.		PHI (23)=	0.1429280E 06
PHI-ACTIVE (24)=0.		PHI (24)=	0.1030530E 06
PHI-ACTIVE (25)=0.		PHI (25)=	0.2189950E 06
PHI-ACTIVE (26)=0.		PHI (26)=	0.2621875E 08
PHI-ACTIVE (27)=0.		PHI (27)=	0.729[illegible]E 07
PHI-ACTIVE (28)=0.		PHI (28)=	0.5708723E 09
PHI-ACTIVE (29)=0.		PHI (29)=	0.2133[illegible]E 07
PHI-ACTIVE (30)=0.		PHI (30)=	0.106[illegible] 07
PHI-ACTIVE (31)=0.		PHI (31)=	0.2253[illegible]E 07
PHI-ACTIVE (32)=0.		PHI (32)=	0.2875[illegible]E 07

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W/WOO=          6.2[illegible]

WOLD-W=   0.28[illegible]19E 02

ABS(POG)=   0.44637[illegible]E-09          P(IPROP)= -0.6250708E-05

RET =   0.4463[illegible]E-09



MINIMUM WEIGHT DESIGN

VR  1 =              143.13031
VR  2 =              255.47041
VR  3 =               15.30000
VR  4 =               63.13446

TOTAL WEIGHT =              6[illegible]28.[illegible]
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AVAILABLE SECTION PROPERTIES

(* AVAILABLE ONLY FOR BEAM MEMBERS)

NO	SHAPES	MP(FT.KIPS)
1	8 WF 20.0	57.3
2	8 WF 24.0	69.3
3	8 WF 28.0	91.3
4	10 WF 25.0	88.5
5	12 WF 27.0	114.0
6	8 WF 40.0	119.7
7	12 WF 31.0	132.0
8	12 WF 36.0	154.2
9	14 WF 34.0	163.5
10	12 WF 45.0	194.7
11	16 WF 40.0	219.1
12	10 WF 60.0	225.3
13	16 WF 50.0	278.1
14	16 WF 58.0	318.0
15	18 WF 55.0	335.0
16	21 WF 55.0	375.0
17	18 WF 64.0	396.0
18	12 WF 99.0	456.0
19	21 WF 68.0	480.0
20	24 WF 68.0	525.0
21	24 WF 76.0	600.0
22	12 WF133.0	630.0
23	24 WF 84.0	672.0
24	27 WF 84.0	729.0
25	12 WF161.0	777.0
26	27 WF 94.0	834.0
27	27 WF102.0	912.0
28	30 WF 99.0	936.0
29	21 WF127.0	954.0
30	21 WF142.0	1071.0
31	30 WF116.0	1134.0
32	33 WF118.0	1242.0
33	24 WF145.0	1243.0
34	27 WF145.0	1356.0
35	33 WF130.0	1398.0
36	33 WF141.0	1539.0
37	33 WF152.0	1674.0
38	36 WF150.0	1740.0
39	36 WF160.0	1869.0
40	36 WF170.0	2001.0
41	36 WF194.0	2301.0
42	33 WF220.0	2508.0
43	36 WF230.0	2829.0
44	36 WF245.0	3024.0
45	36 WF260.0	3228.0
46	36 WF280.0	3501.0
47	36 WF300.0	3765.0

	CONTINUOUS SOLUTION	AVAILABLE SOLUTION
MP 1(FT,KIPS)	142.131	143.510(14 WF 34.0...SELECTED FROM AVAILABLE WF AND B SHAPES AS A BEAM MEMBER)
MP 2(FT,KIPS)	[illegible]	276.100(15 WF 50.0...SELECTED FROM AVAILABLE WF AND B SHAPES AS A BEAM MEMBER)
MP 3(FT,KIPS)	16.700	57.300(9 WF 20.0...SELECTED FROM AVAILABLE WF SHAPES AS A COLUMN MEMBER)
MP 4(FT,KIPS)	62.157	69.300(9 WF 36.0...SELECTED FROM AVAILABLE WF SHAPES AS A COLUMN MEMBER)
OBJ. FUNCTION	6.225	7.014
TOTAL WEIGHT (KIPS)	6.225	6.840