1975

Plastic design of unbraced multistory steel frames, DRAFT, June 1975

George C. Driscoll Jr.

Koichi Takanashi

Reinhard L. Gsellmeier

Paul W. Reed

DRAFT

PLASTIC DESIGN OF UNBRACED MULTISTORY STEEL FRAMES

by

George C. Driscoll, Jr.

Koichi Takanashi

Reinhard L. Gsellmeier

Paul W. Reed

This research has been carried out as part of an investigation sponsored by the American Iron and Steel Institute, American Institute of Steel Construction, and National Science Foundation.

Fritz Engineering Laboratory
Department of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania 18015 U.S.A.

June 1975

Fritz Engineering Laboratory Report No. 367.12

CHAPTER 1

Introduction

1.1 OBJECTIVE

The objective of this publication is to acquaint practicing engineers with the present state of the theory for the plastic design of ~~braced~~ unbraced multistory steel frames. It is hoped that the information presented will stimulate the use of plastic design methods for frames of this type, and that this in turn will produce an input of useful ideas contributing to the full development of the concept.

1.2 CONTENTS

The information contained herein is mainly a digest of the research material presented to engineering educators at the Lehigh University Conference on Plastic Design of Multistory Frames[1] in August 1965. An effort has been made to include enough theory for the engineer to understand the behavior of the structure but to concentrate principally on design aspects. The engineer who wishes to delve into the background of research should study the references listed. The design example of ~~a braced~~ an unbraced multistory frame will serve as a guide to the efforts of the practicing engineer as he applies the principles of plastic design to his own work. The grades of steel used in the design example are A36 with F_y = 36 ksi and ~~A441 or~~ A572 with F_y = 50 ksi. Design aids for these values are included. A listing of the notation used is given for ready reference. Sign conventions are discussed as they are developed.

1.3 THE FUTURE OF MULTISTORY FRAMES

Multistory and high-rise buildings have been common in some of our nation's large cities, but recent sociological trends have forced the use of such structures in more numerous locations and have pushed them to even greater heights. As the population increases and tends to concentrate in urban areas, and as land costs skyrocket, the multistory building becomes the economical solution to housing people for living and working. The tall building will be the common structure of the future and economy of the structural frame is of increasing importance. Structural steel frames proportioned by plastic design methods may offer savings over frames of other materials and over steel frames designed by allowable stress methods.

Often a braced frame will prove to be the most economical solution to a multistory frame design, and bracing should be used whenever feasible. The choice of an unbraced frame is frequently dictated by a desire for large clear spaces or openings. Some sacrifice in framing economy must be expected when an unbraced moment-resisting frame is selected. Design of tall structures for resistance to earthquake requires ductile moment resisting frames to be provided. Plastic design methods should prove to be useful in proportioning such frames. Research is being conducted to further improve the knowledge about behavior of frames inelastically deformed by earthquake forces.

1.4 THE DESIGN TEAM

Regardless of the design method, the building process today demands an integrated team of architects, and electrical, mechanical and structural engineers. Each must understand the other's requirements, for rising costs and increased demand for excellence in construction require the integration of all building components into a compact structure with a minimum of wasted volume. The structural engineer must understand the architect's desire to have the structural frame complement the function and the motif of the building. He must be appreciative of the space needed for the conduits and ducts required by the electrical and mechanical engineers as they attempt to regulate the internal environment of the modern building. Within such constraints he must produce a safe and economical structural frame. The frame must safely support the gravity and wind loads without undue deflection or sway affecting the operation of other building components or producing unpleasant sensations to the occupants.

Further details on the problems and interactions of all members of the design team along with exhaustive lists of references are presented in a series of state-of-art reports contained in Reference

1.5 NEW STRUCTURAL CONCEPTS

Fortunately, the structural engineer is assisted in fulfilling these requirements by new knowledge of how structures behave, and by the advent of new materials, products and construction techniques. Research on the behavior of steel structures during the last twenty years has led to the development of the plastic design philosophy as contrasted to the more established methods of elastic design, more correctly known as allowable stress design. Composite design uses the integrated strength of steel and concrete. New high strength structural steels of carbon, low alloy and heat treated types permit a reduction in the sizes of members. High strength bolts and new welding techniques produce economical, rigid connections of greater compactness and more direct transfer of stress.

1.6 ALLOWABLE STRESS DESIGN

The current method of designing rigid multistory building frames [2] involves the determination of the internal shears, moments and thrusts caused by working loads using methods of allowable stress analysis for statically indeterminate structures. Because of the high order of redundancy of the multistory rigid frame the analysis is usually reduced to a statical one by making appropriate assumptions as in the "portal" or "cantilever" methods. Using the internal forces and an allowable stress, derived principally by dividing the yield point stress of the steel by a factor of safety, the members are proportioned using ordinary mechanics of materials equations. Inherent in this approach is the philosophy that the limit of usefulness of the structure is reached as soon as the yield point stress is developed at one point in the frame. Other points in the frame will be understressed, and thus uneconomical in the use of material. This method does not recognize that local yielding in a rigidly connected steel structure permits a redistribution of the internal forces to less highly stressed parts of the structure, and consequently it underestimates the load carrying capacity of the structure as a whole. Local yielding is not detrimental to the behavior of the structure provided it is contained by adjacent elastic regions of the frame.

1.7 PLASTIC DESIGN

On the other hand, the plastic design philosophy recognizes the redistribution of internal forces that takes place when complete yielding (plastic hinges) develops at regions of high bending moment. It focuses on the limit of usefulness as the ultimate load that can be carried just before the structure develops a sufficient number of plastic hinges to permit unrestrained deformation of the structure. This ultimate load is an indication of the strength of the whole structure, and it exceeds the working load by a factor F. The quantity F, called the load factor, is selected to be consistent with the factors of safety inherent in the allowable stress design of a simply supported beam. In this publication the following values, adopted from the Lehigh Conference, are used for beams, columns and frames:

Gravity loading $F = 1.70$
Gravity and wind loading $F = 1.30$
Gravity and earthquake loading F = 1.30

Uncertainty about stability problems was the chief reason for a somewhat higher load factor specified for frames in the past.[3] New research presented at the Lehigh Conference has led to a better understanding of the behavior of columns and therefore the values of F shown appear justified.

Deflection may also constitute a limit of usefulness for the structure, and whether designing by allowable stress or plastic methods, it is necessary to consider the vertical beam deflections and horizontal frame deflections (drift) under working loads. Deflections rather than strength may actually govern the design.

Perhaps the future may see an evolution of the best features of both allowable stress design and plastic design into a structural design satisfying performance requirements.

CHAPTER 2

Dimensions and Loading

2.1 CHOICE OF DIMENSIONS

The overall dimensions of the multistory building are governed by the size and shape of the site available and by set-backs from the property lines required by zoning ordinances. For reasons of architectural layout it is often advantageous for the building to be long and narrow. Within these area limitations it is the responsibility of the architect-engineer design team to determine the required number of floors to fulfill the owner's space needs. Many municipalities have zoning ordinances restricting heights of buildings, but these restrictions are being removed or liberalized as codes are revised.

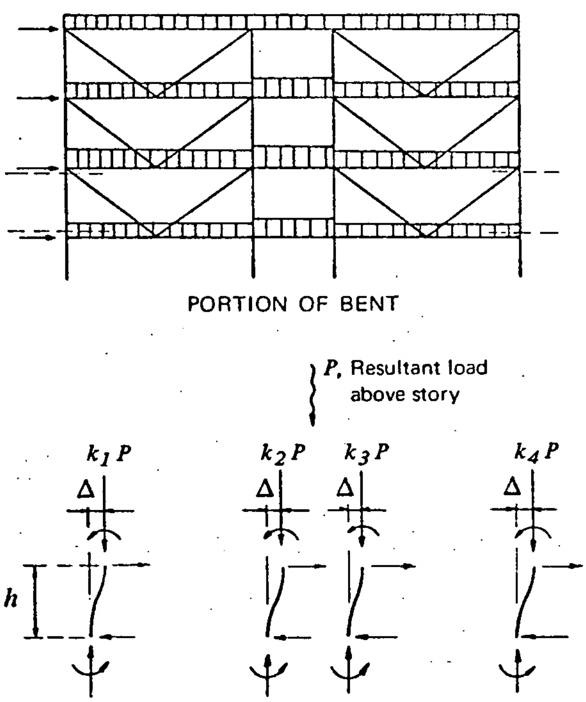
The design team must decide on bay sizes for the structural frame that fit the architectural and mechanical-electrical layouts of the integrated structure. There is a trend toward the use of larger bay dimensions, particularly with composite floor beams. Longer spans increase the depth of the floor system, thereby increasing the height of the building. However, increased floor depth often permits more economical construction even though the building volume is increased.

Regardless of the method of structural design, the items mentioned above must be considered and examined from their technical and economical aspects before bay sizes are established. The bay sizes shown for the apartment house example of Chapter ~~8~~ 9 represent a possible, but not necessarily the best, framing plan for that structure. They represent a compromise based on the integrated requirements.

2.2 BRACING METHODS

The multistory building must be designed to provide resistance to horizontal forces applied in any direction. A number of devices may be used, including shear walls or core sections, but in the
example in Chapter 8 attention will be directed 9
toward proportioning of the steel bents to provide the necessary strength and limitation to drift. There are two conventional methods of providing the necessary resistance.

One or more bents of a frame may be braced for the full height of the building using diagonal or K-bracing. This creates a vertical cantilever truss to which all wind load is transmitted. In the allowable stress design of this type of framing the girders may have either simple or rigid connections to the columns. Plastic design requires rigid connections. Rigid connections have real advantages in allowable stress design also. For example, rigidly connected members reduce beam deflections, reduce beam depth, and reduce floor cracking.



ISOLATED ONE-STORY COLUMNS

FIG 2.1 THE PΔ EFFECT DUE TO SWAY

On the other hand, resistance to the horizontal forces may be provided entirely by the bending resistance of rigidly connected girders and columns.

It is desirable to define braced and unbraced bents in terms of the method of resisting secondary moments produced by drift. When a building drifts, each floor moves laterally with respect to the adjacent floors as indicated in Fig. 2.1. The vertical forces kP on the columns at one floor become eccentric with respect to the column axes at the floor beneath by an amount Δ, producing secondary moments totaling $P\Delta$.

In this publication the following definitions and assumptions will be used:

Braced Bent — Has physical brace in at least one bay of a bent on each floor. $P\Delta$ effect is controlled by the shear resistance of the bracing system. Girder connections ~~are rigid~~ may be rigid or simple.

Unbraced Bent — No physical brace. Strength depends on bending resistance of all members. $P\Delta$ effect must be resisted by the columns in bending. Girder connections are usually rigid. Some simple connections may be used in each floor provided the remaining rigid connections have ample strength and stiffness.

Supported Bent — Depends on adjacent braced or unbraced bents for resistance to horizontal forces and $P\Delta$ effects; is designed for gravity loads only. Girder connections ~~are rigid~~ may be rigid or simple.

2.3 GRAVITY LOADS

Building codes specify the working live loads for floors, the roof load and wind loads. The dead load, floor live loads and roof loads are referred to as gravity loads. Although the dead load is always present many variable patterns of live loading are possible. Codes[4] permit a reduction in the live load for beams or girders supporting large floor areas and for columns supporting several tiers of floors. Such reductions recognize the improbability of having the full live load acting over large areas and on all floors simultaneously.

Partial live loading in a checkerboard pattern may control the column design. Checkerboard loads produce a lower axial force in the columns but may produce a more critical bending effect.

2.4 HORIZONTAL LOADS

Wind loads are usually expressed as a resultant unit pressure applied horizontally against the windward side of the building. Many modern codes require an increase in wind pressure as the height above the ground increases. It is customary to convert the wind pressure to forces applied at each floor level, and to assume that the floors, acting as diaphragms, transfer the wind forces along the building to the periodically spaced braced frames.

The application of plastic design to seismic loading is an area of current study.[5] Some methods of design against seismic loads apply a static lateral force calculated as a certain percentage of the weight of the building and then use the procedure established for design against wind. Use of this procedure will be illustrated in examples.

2.5 INSTABILITY OF BRACED FRAMES

Instability is a phenomenon which may occur either for an individual member in a frame, for an entire story within a frame, or for a whole frame. Member instability due to local buckling prior to the attainment of the member plastic moment capacity is avoided by selecting only sections which are specified as compact by the AISC Specification as explained in Art. 3.5a. Member instability due to lateral-torsional buckling of beam-columns is avoided by designing members in accordance with the stability interaction equation of Section 2 of the AISC Specification, as explained in Art. 3.5b. Instability of an entire story within a frame may occur under either gravity loads alone or under combined gravity plus lateral loads. Frame instability under these loadings will be examined here.

2.5a GRAVITY LOAD INSTABILITY

Theoretically a symmetric frame under symmetric gravity loads will not sway as the loads are increased. The gravity loads will continue to increase until at some given load value called the frame buckling load the frame will pass from a symmetric stable configuration to an unsymmetric unstable configuration characterized by a large lateral deflection (Fig. 2.2a). This type of failure is called frame buckling, and is purely theoretical in an actual frame.

INSERT FIG. 2.2

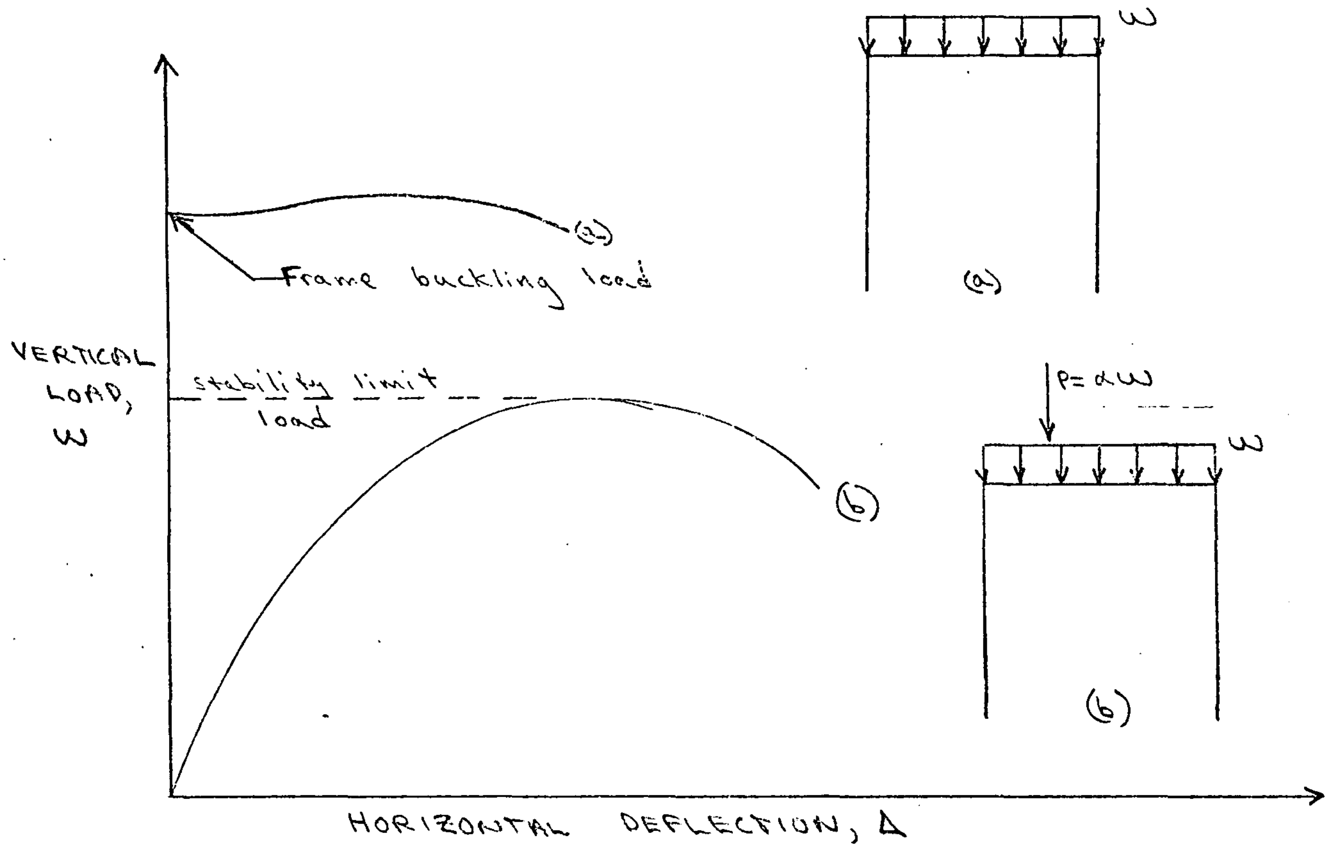


Fig.2.2– Frame buckling (a) and frame instability (b) under gravity loads.

In an actual building frame, eccentricities exist which from the start give a horizontal deflection upon application of any vertical load. Such is also the case for an unsymmetric frame, or a symmetric frame under unsymmetric gravity loads. As the loads increase the drift increases, giving rise to secondary $P\Delta$ overturning moments, which cause the drift to increase even more rapidly. At a certain value of load called the stability limit load, the frame will continue to sway without further increase in load (Fig. 2.2b). This type of failure is called frame instability.

For most frames of usual dimensions, the upper 4 to 6 stories are controlled by gravity loads alone, whereas the lower and middle stories are controlled by combined gravity plus lateral loads. For this reason, instability under gravity loads will only be a problem in the upper stories of a frame, the lower and middle stories all being overdesigned for the load case of gravity loads alone.

Recent research results (Ref. 1) indicate that for most frames, frame instability due to gravity loads in the upper stories will very rarely preclude the attainment of the ultimate factored gravity loads. This is especially the case for columns which are designed by the provisions of Chapter 3, in which column axial loads are limited to .75 Py. Gravity load instability is also prevented in the upper stories by the stiff base support which these stories receive from the middle and lower stories (Ref. 2). For these reasons, gravity load instability can be neglected for typical frames.

There are certain special cases in which the gravity load instability problem may be further investigated according to the procedures given in

Ref. 3. Fig. 2.3 illustrates some of these special cases for which gravity load instability may in fact govern the design of the frame.

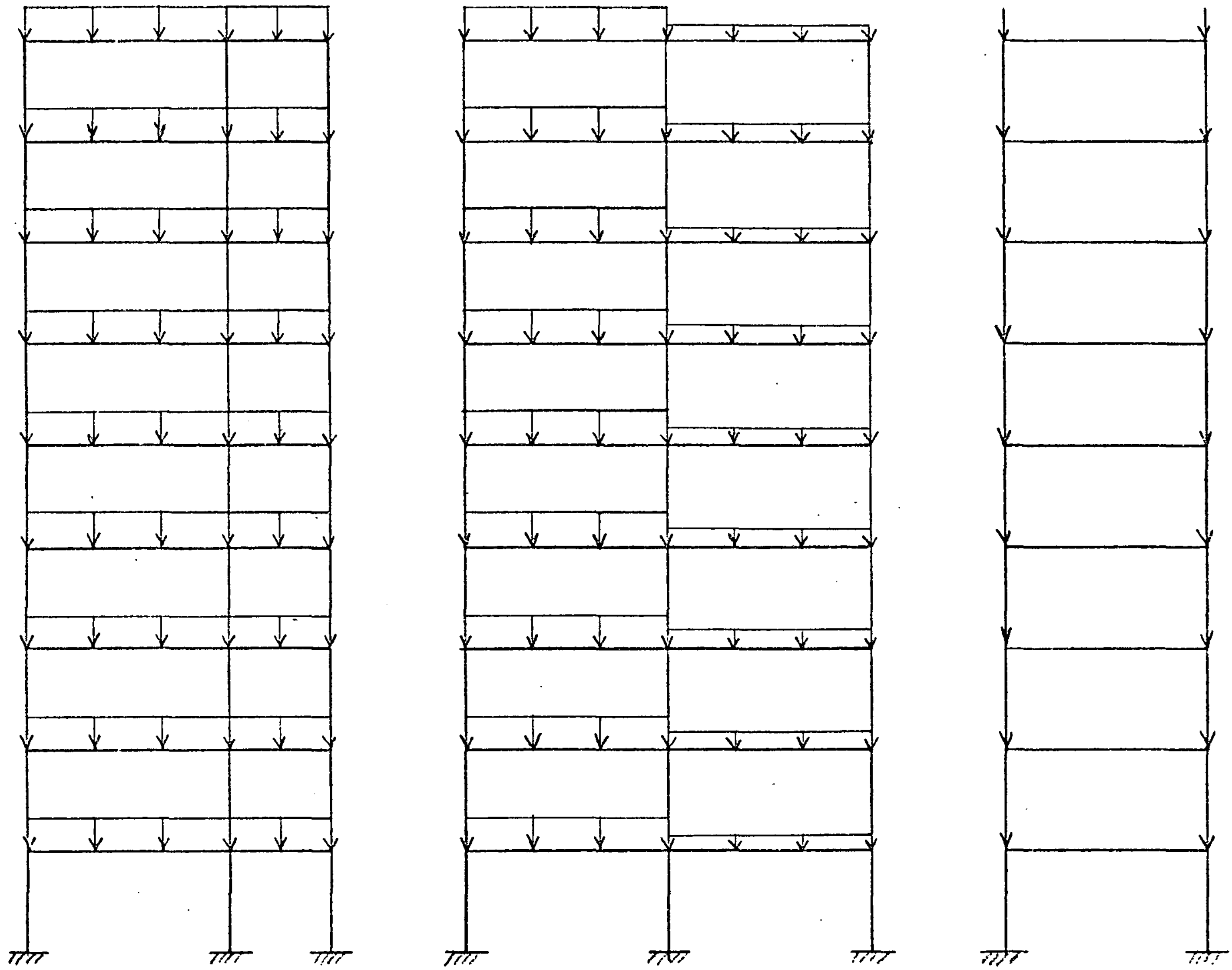
INSERT FIG. 2.3

2.5b COMBINED LOAD INSTABILITY

As stated in ASCE's Manual No. 41, "Plastic Design in Steel", pps. 240-241 (Ref. 4):

> "In the more general case an unbraced frame will resist combined gravity and lateral loads, but at a lower load factor. For unbraced frames subjected to combined loads, later deflections will occur from the first application of load. Initially the loads and resulting lateral deflections will be nearly proportional. As the applied loads increase, however, PΔ effects and yielding will cause the lateral deflections to increase at a greater rate than the rate of loading until, at the stability limit load, the frame will continue to sway without further increase in the load (Fig. 2.4). This type of frame behavior is also called frame instability."

INSERT FIG. 2.4



(a) Highly unsymmetric frames.

(b) Unsymmetric gravity loads

(c) Loads transferred to columns directly by spandrels.

Fig. 2.3 - Specials cases in which gravity load instability may control the design of the upper stories.

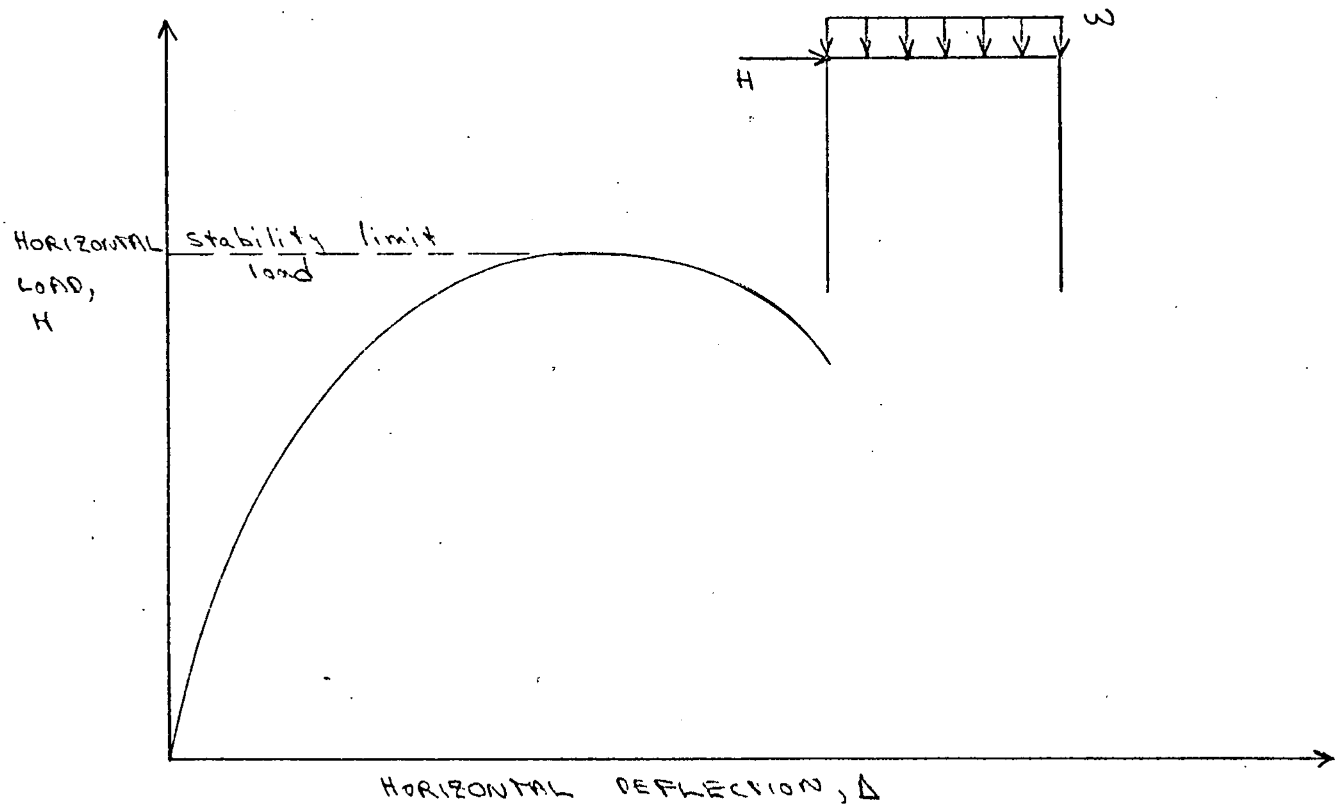


Fig. 2.4 - Frame instability under combined loads.

To include the effect of frame instability under combined loads, the $P\Delta$ overturning moments must be included in the preliminary design. A very large, conservative value of Δ is usually estimated for preliminary design purposes. After preliminary member sizes have been selected, several stories in the frame should be checked by a horizontal load versus drift analysis (Chapter 7) to determine the working load deflections, and the maximum strength of the story.

CHAPTER 3

Fundamentals of Plastic Design

3.1 MATERIAL PROPERTIES

The successful application of plastic design to structures depends on two desirable properties of structural steel—strength and ductility. These are portrayed by the stress-strain diagram (Fig. 3.1). The level of strength used in plastic design is that of the yield plateau, F_y. The length of

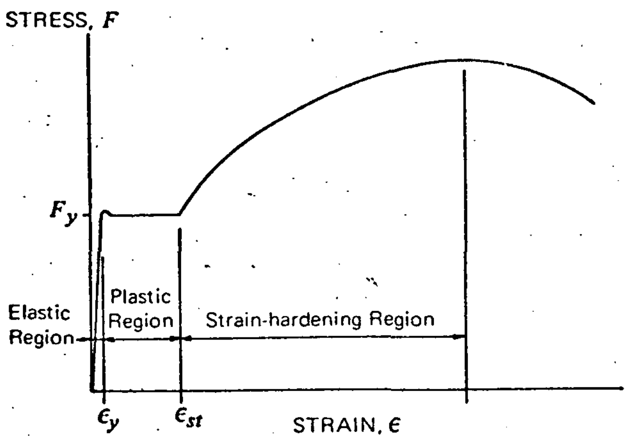


FIG 3.1 STRESS-STRAIN DIAGRAM FOR STRUCTURAL STEEL

that plastic plateau is a measure of the ductility; for A36, A441, and A572 steels the strain at the limit of the plastic region, ϵ_{st}, is approximately

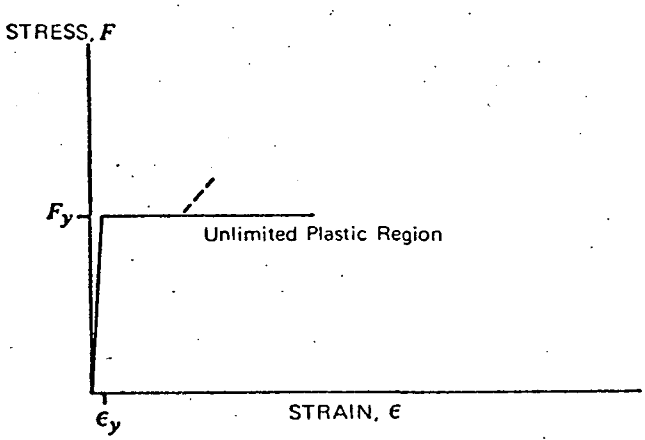


FIG 3.2 IDEALIZED STRESS-STRAIN DIAGRAM

12 times the strain at the initiation of yielding, ϵ_y. In plastic design the actual stress-strain diagram is replaced by an idealized diagram representing steel as an elastic-plastic material (Fig. 3.2).

The allowable stress design method defines the limit of usefulness of a cross-section as occurring when the strain in one fiber only reaches ϵ_y, but the plastic design method considers the remaining usefulness after the attainment of ϵ_y in all fibers. That is, the cross-section becomes fully plastic (Fig. 3.3).

3.2 IDEALIZED CONCEPTS FOR BEAMS

Plastic design has its chief utility in the design of structures composed of bending members. In such members the strains are proportional to the distance from the neutral axis under all magnitudes of loading but the stresses are not proportional once the fibers have strained beyond ϵ_y. When the bending moment at a section becomes so great that practically all fibers have strains greater than ϵ_y, the stress distribution diagram approaches a fully yielded condition known as a

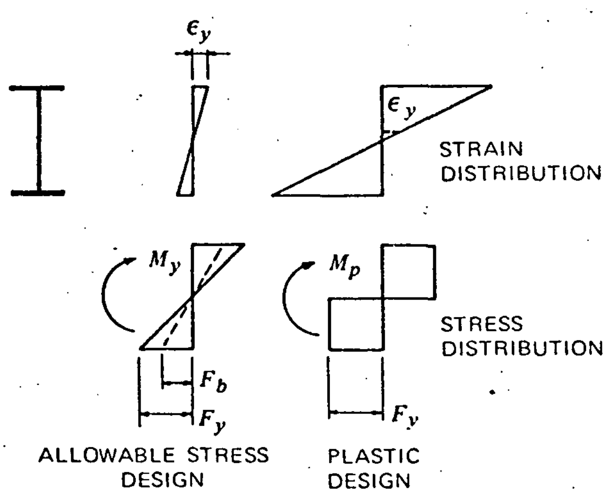


FIG 3.3 LIMIT OF USEFULNESS, BENDING ONLY

plastic hinge. The plastic hinge is a condition of limiting moment resistance at that cross-section of the beam. Increases in load produce greater strains but the moment remains constant at the plastic moment, $M_p = F_y Z$. Z is the plastic section modulus, a geometric property of the cross-section that may be found in handbooks.[3]

When the full cross-section of a wide flange or I-beam becomes plastic, the resisting moment M_p is about 12% greater than the moment that causes first yielding, M_y.

$$\frac{M_p}{M_y} = \frac{F_y Z}{F_y S} = \frac{Z}{S} = \text{shape factor} \cong 1.12 \qquad (3.1)$$

For simple span beams this is the only gain in load carrying capacity arising from the plastification of the cross-section and, therefore, provides little incentive to use plastic design procedures. However, in continuous structures the formation of a plastic hinge at one location changes the restraint characteristics of the structure and a redistribution of internal forces takes place. The redistribution permits other cross-sections to operate to their full strength so that the overall load carrying capacity of the structure is utilized when the limiting load, P_u, is reached.

The limiting load for a beam is the lowest value of load that will produce enough plastic hinges for a plastic mechanism to form. A plastic mechanism is similar to a mechanical linkage except that the elastic portions of the structure are connected by plastic hinges rather than frictionless real hinges. Under this condition appreciable deflections may occur but continued deflection is restrained by the advent of strain hardening. Furthermore, the limiting load is not the true ultimate load for the structure because the steel has an ultimate strength greater than yield strength. However, this reserve is not fully realized if prior local or general instability conditions develop.

There are a number of independent types of plastic mechanisms but in multistory buildings the important ones are the beam type occurring under gravity loads alone, the panel type occurring under wind loading alone, and a combination of the beam and panel mechanism under the combined loading. The actual location of the hinges depends on the loading and the relative strength of the girders and columns. The mechanisms shown in Fig 3.4 assume relative sizes that cause hinges to form in the girders.

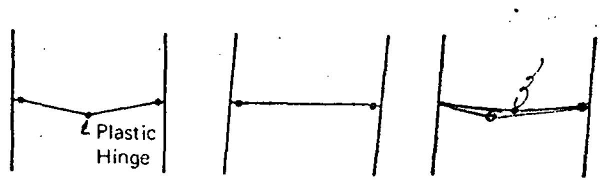


FIG 3.4 SOME PLASTIC MECHANISMS

In the panel type mechanism and combined mechanism two characteristics of the beam behavior are of special interest in dealing with unbraced multistory frames. These are the moment diagram and the beam stiffness. The manner in which moments and stiffness change during progressive plastification of the beams must be understood in order to design a frame for strength and resistance to drift.

The simple beam moment diagram of Fig. 3.5a has a shape and magnitude governed by the size and location of loads and by the span. The same relative size and shape of moment diagram is maintained even when the beam is built into a structure causing end moments as shown in Fig. 3.5b. Provided that no plastic hinge has formed under gravity load alone, the beam will be able to participate in frame resistance to lateral load. Initially the beam will exhibit normal elastic stiffness. With a lateral force from the left on the frame, both ends of the beam will be forced to rotate clockwise and moment changes resembling Fig. 3.5c will occur. Both end moments will be functions of both end rotations. For the special case where both rotations θ are equal, the changes in moment are

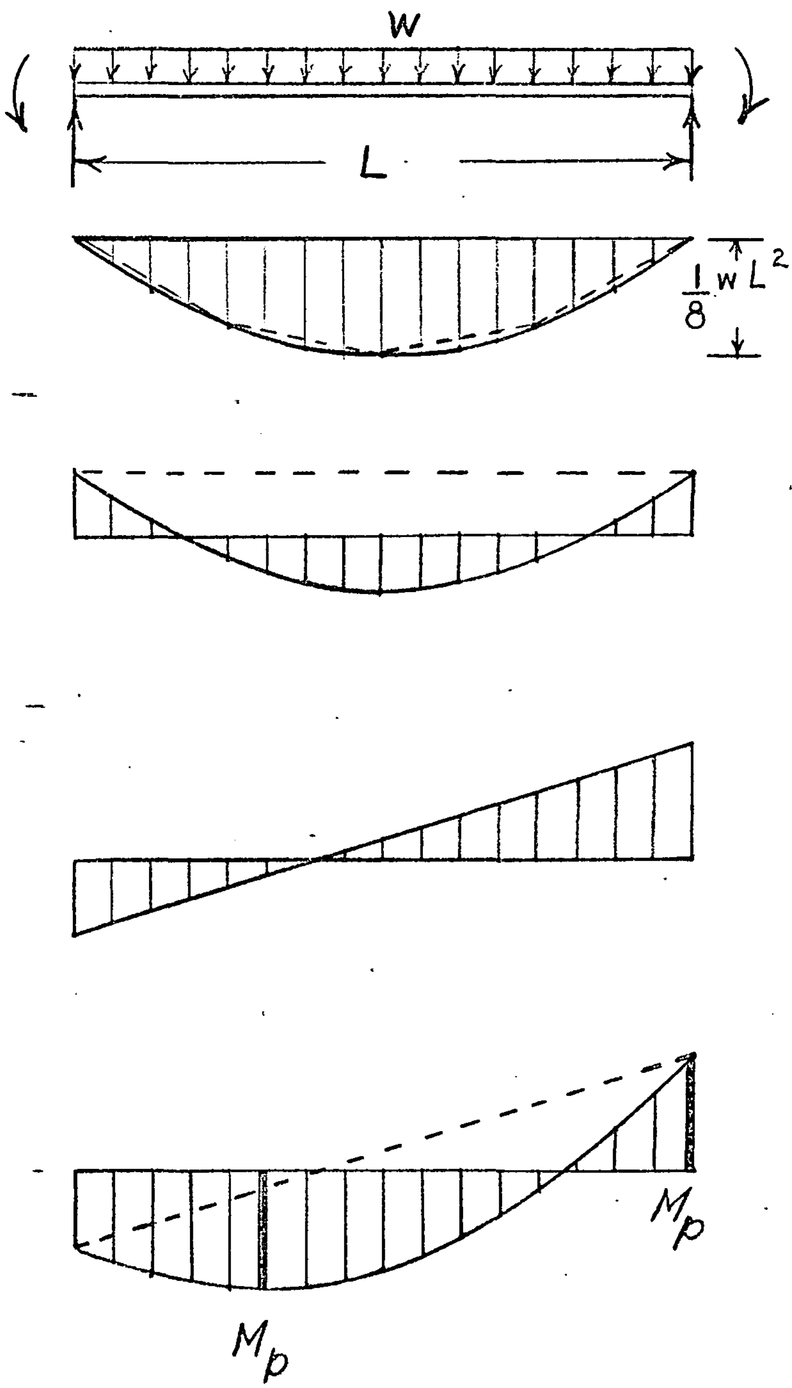
$$\partial M = 6 \frac{EI}{L} \partial\theta \qquad (3.2)$$

The term 6 EI/L is the stiffness of the beam against drift. Typically, the wind moment superimposed upon the initial moment at the lee end of the beam will cause a plastic hinge to form. Then the beam will be unable to accept any increase in moment at its lee end although the plastic hinge moment M_p will be maintained. The stiffness at the lee end will reduce to zero but the windward end will still be able to accept increases in moment with a stiffness reduced by one half

$$\partial M = 3 \frac{EI}{L} \partial\theta \qquad (3.3)$$

This behavior can continue until a second plastic hinge forms somewhere between the center and the Windward end of the beam giving a limiting moment diagram such as Fig. 3.5d. From this stage on the beam has no end stiffness at either end and cannot participate in resistance to frame drift.

INSERT FIG. 3.5



Load pattern

(a) Simple beam moments (Parabola or polygon)

(b) Built-in beam moments

(c) Wind moments with wind from the left

(d) Combined wind and vertical load moments

Fig 3.5 Characteristics of Moment Diagram for a Beam

The complete limiting moment diagram of Fig. 3.5d can be determined by equilibrium for a given span, M_p and loading. The solution can be presented in equation form or chart form for use in preliminary design or in analysis by the moment balancing method. The limiting moments along with the stiffness characteristics may be used in the sway subassemblage method to analyze for drift. From such studies it is found that the greater the excess of beam capacity over that needed for gravity load alone, the greater its ability to assist the frame in resisting lateral force, frame instability and drift.

For a plastic mechanism to develop, the first hinge to form must be able to rotate at a nearly constant moment, M_p, until the last hinge develops. In other words, first formed hinges must possess rotation capacity. A way of showing the ability of a beam to carry moment during rotation is by a moment-curvature graph
(Fig. ~~3.5~~). 3.6

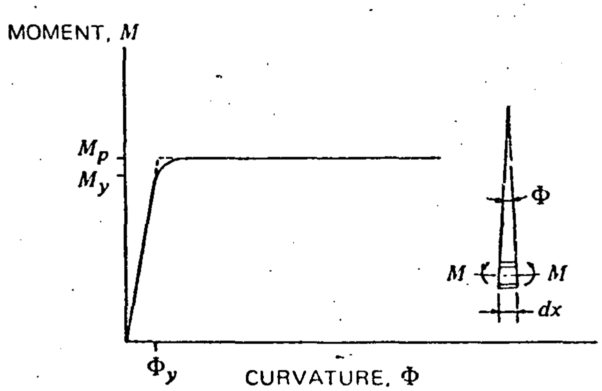


FIG ~~3.5~~ MOMENT-CURVATURE GRAPH 3.6

3.3 MODIFYING FACTORS FOR BEAMS

Anything that interferes with the rotation of the hinge or reduces the moment capacity of the hinge causes a deviation of the actual behavior of the beam from that predicted by simplified plastic design theory.

Factors which may cause deviations from idealized plastic hinge behavior are:

1. local buckling
2. lateral-torsional buckling
3. shearing force
4. axial force

3.3a LOCAL BUCKLING IN BEAMS

The development of the plastic moment and of adequate rotation may be prevented by localized buckling of the compression flanges. To ensure adequate hinge rotation the width, b, and thickness, t, of the beam flange must be such that the flange can compress plastically to strain hardening, ϵ_{st}, without buckling. The web of a beam, which is partially stressed in compression due to flexure, is also prone to local buckling if the ratio of web depth, d, to thickness, w, is too large. Limiting d/w to a specified value will prevent local web buckling of a beam subjected to bending only. However, if axial force is combined with plastic bending moment a reduction in the permissible d/w ratio must be made. See Article 3.4b.

The limiting ratios of flange and web dimensions to inhibit local buckling of beams are tabulated in Table 3.1.

TABLE 3.1

Specified Minimum Yield Point, F_y	Flange b/t	Web d/w
36 ksi	17.4	70
50 ksi	14.8	60

3.3b LATERAL-TORSIONAL BUCKLING IN BEAMS

Another type of buckling must also be prevented in order to ensure satisfactory performance of a plastically designed beam.

When an I-shaped beam bends about its strong axis it may buckle out of the plane of bending. The deflection consists of a lateral movement of the compression flange and a lesser movement of the tension flange so that twisting of the section occurs (Fig. ~~3.6~~). This lateral-torsional buckling,

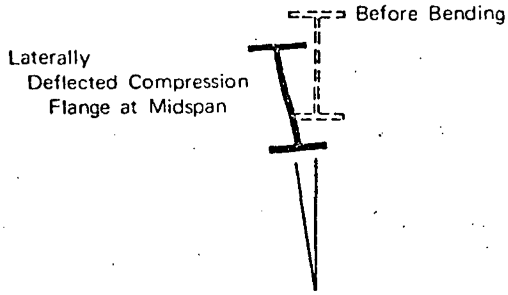


FIG ~~3.6~~ 3.7 LATERAL TORSIONAL BUCKLING
OF A COMPACT BEAM

is more general in nature than local buckling, affecting larger regions of the beam. It is important that members be adequately braced because failure to provide adequate bracing, particularly at plastic hinge locations, may preclude full hinge rotation and the development of a plastic mechanism. The same tendency toward lateral buckling occurs in members designed by allowable stress methods, but the problem is less critical for it is not required to guarantee the development and rotation of a plastic hinge.

Lateral-torsional buckling develops more readily in segments of the beam where the bending moment is almost constant than in segments having a steep moment gradient. Thus,

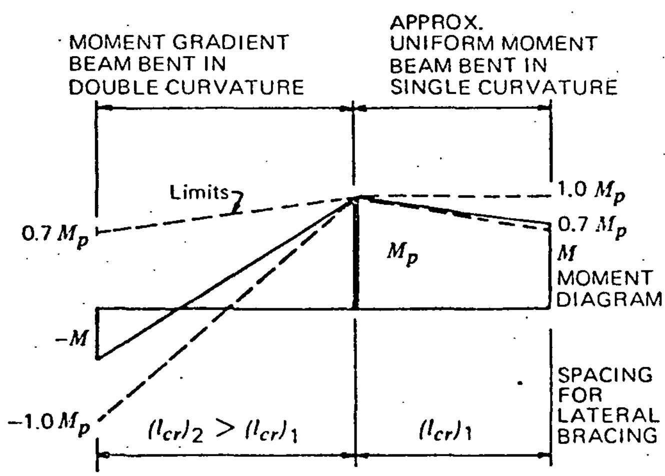


FIG ~~3.7~~ 3.8 BRACING LOCATIONS FOR BEAMS

the rules for spacing of lateral bracing provide for a variable distance, l_{cr}, depending upon the ratio of the moment, M_p, at the braced hinge and the moment, M, at the other end of the unbraced segment (Fig. ~~3.7~~ 3.8).

Recent analytical work [1] taking into account different kinds of steels and the stress condition of the adjacent segments, justifies the provisions tabulated in Table 3.2 for l_{cr} with the common condition of elastically stressed adjacent segments.

Specified Minimum Yield Point, F_y	$(l_{cr})_1$ Uniform Moment $-0.5 \geq \frac{M}{M_p} \geq -1.0$	$(l_{cr})_2$ Moment Gradient $1.0 \geq \frac{M}{M_p} \geq -0.5$
36 Ksi	$38\ r_y$	$63\ r_y$
50 Ksi	$28\ r_y$	$53\ r_y$

If a braced segment, l, of a beam is bent about its strong axis by equal end moments causing uniform moment the end moments will reach M_p provided $l \leqslant l_{cr}$. However, if $l > l_{cr}$ lateral-torsional buckling will occur at $M_m < M_p$ as

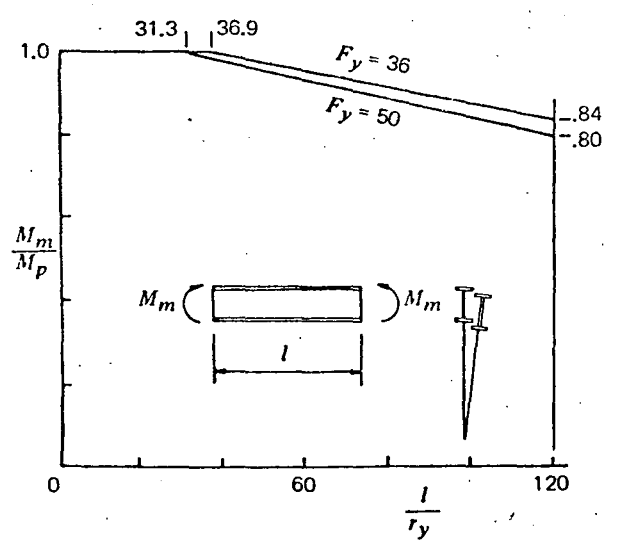


FIG ~~3.8~~ LATERAL TORSIONAL BUCKLING UNDER UNIFORM MOMENT 3.9

shown by Fig ~~3.8~~. This value of M_m is of importance in the lateral-torsional buckling of beam-columns. 3.9

In segments where the beam is behaving elastically or at the last hinge of the plastic mechanism the spacing of braces is determined by rules of allowable stress design. Recommendations for sizes of lateral braces are given in Ref 1.

3.3c SHEARING FORCE IN BEAMS

The simplified plastic theory is developed for conditions of pure bending but in practice flexure is usually accompanied by shearing forces. The influence of shear is masked by strain hardening and local and lateral buckling, but, as a design criterion, the limiting shear may be taken as the force that causes the entire web to yield in shear, V_u. Beams and columns should be proportioned according to

$$V \leqslant V_u = 0.55F_y wd \qquad (\cancel{3.2}) \quad 3.4$$

where F_y is in ksi.

If V exceeds the shear carrying capacity of the beam, V_u, a new beam with greater web area may be chosen, or the web may be reinforced with doubler plates.

3.4 AXIALLY LOADED COLUMNS

Columns in multistory building frames will usually be subjected to axial force and bending moments. The column will be loaded by axial force alone only if the shears and end moments from the girders are symmetrical about the column centerline at a particular floor level.

The maximum strength of an axially loaded compression member may conservatively be estimated as

$$P_{cr} = 1.7\ A\ F_a \qquad (3.5)$$

where A is the gross area of the member, and F_a is the allowable stress given by Formula (1.5-1) of the AISC specification.

$$F_a = \frac{\left[1 - \dfrac{\left(K\dfrac{h}{r}\right)^2}{2\ C_c^2}\right] F_y}{\text{F.S.}} \qquad (3.6)$$

for $K\frac{h}{r} \leq C_c$, where $C_c = \dfrac{23{,}900}{\sqrt{F_y}}$

Building columns usually have slenderness ratios less than C_c and failure will occur by inelastic buckling.

However, for design purposes in unbraced frames, the maximum load should be limited to

$$P_{max} \leq 0.75\ A\ F_y \tag{3.7}$$

The factor of safety, F. S., is a variable quantity ranging from 1.67 to 1.92. The limitation of $.75AF_y$ is to safeguard against the loss of stiffness due to residual stress, and to prevent extensive yielding of the column ends at the factored load.

For columns in plastically designed unbraced frames an effective length factor of K = 1 can be used provided the secondary PΔ moments are included in the design (Art. 4.1). The slenderness ratio then becomes h/r, and the appropriate ratio to use is given by Table 3.3

TABLE 3.3 - SLENDERNESS RATIOS

	Bending About Strong Axis		Bending About Weak Axis
	Braced Columns	Unbraced Columns	
P_{cr}	$\frac{\ell}{r_x}$	$\frac{\ell}{r_y}$	$\frac{\ell}{r_y}$
P_e	$\frac{\ell}{r_x}$	$\frac{\ell}{r_x}$	$\frac{\ell}{r_y}$
M_m	M_{px}	Eq.3.14, $\frac{\ell}{r_y}$	M_{py}

A column is considered to be fully braced if the slenderness ratio ℓ/r_y is less than ℓ_{cr}, where ℓ_{cr} is given by the equations

$$\frac{\ell_{cr}}{r_y} = \frac{1375}{F_y} + 25 \text{ when } +1.0 \geq \frac{M}{M_{px}} > -0.5 \tag{3.8}$$

$$\frac{\ell_{cr}}{r_y} = \frac{1375}{F_y} \quad \text{when } -0.5 \geq \frac{M}{M_{px}} \geq -1.0 \tag{3.8a}$$

where M is the lesser of the end moments, and M/M_{px} is positive for reverse curvature, negative for single curvature.

3.5 BEAMS - COLUMNS

A column in a multistory building frame will have to resist both an axial load and end moments if the shears and end moments from the girders are not symmetrical about the column centerline at a particular floor level. Such a member is termed a beam-column. In most unbraced frames, the vertical members are beam-columns. The design of beam columns for strength and stability will be considered separately here.

3.5a STRENGTH OF BEAM COLUMNS

The ultimate strength of a beam-column depends on:

1. the member cross-sectional properties

2 → ~~1.~~ the material properties, expressed by F_y

3 → ~~2.~~ the slenderness ratio, h/r

4 → ~~3.~~ the axial load ratio, P/P_y

5 → ~~4.~~ the magnitude of upper and lower end moments, M_U and M_L, respectively

6 → ~~5.~~ the direction of the end moments expressed by q, the ratio of the numerically smaller to the numerically larger end moment

For very short beam-columns failure in a buckling mode is precluded, and the plastic hinge develops at a reduced plastic moment value designated as M_{pc}. For a given value of axial load, M_{pc} depends only on the cross-sectional properties of the member, and the yield stress of the steel. The influence of the axial force in reducing the value of M_{pc} is seen in Fig.3.10 for both strong and weak axis bending of W-shapes.

Strong Axis Bending:

$$M_{pc} = M_{px} \qquad 0 \le P \le 0.15\, P_y \qquad (3.9)$$

$$M_{pc} = 1.18\left(1 - \frac{P}{P_y}\right) M_{px} \qquad .15P_y \le P \le P_y \qquad (3.9a)$$

Weak Axis Bending:

$$M_{pc} = M_{py} \qquad 0 \le P \le 0.4\, P_y \qquad (3.10)$$

$$M_{pc} = 1.19\left[1 - \left(\frac{P}{P_y}\right)^2\right] M_{py} \qquad 0.4\, P_y \le P \le P_y \qquad (3.10a)$$

INSERT FIG. 3.10

It is emphasized that M_{pc} is a basic characteristic of a short compression member and is not indicative of the carrying capacity of longer beam-columns where the slenderness ratio h/r will have an appreciable influence on the behavior of the member, as demonstrated below.

The ultimate strength of beam-columns, including the effect of slenderness ratio, may be represented by moment-rotation curves or by interaction curves. Both procedures will be described briefly.

The effect of the magnitude of the axial load on a short column's ability to resist moment has been illustrated in Fig ~~3.9~~ 3.10. Another way of showing this is by *M-P-Φ* diagrams as plotted in Fig ~~3.10~~ 3.11 for a particular size column. This plot shows the influence of the axial load in reducing the moment carrying capacity of ~~an 8WF31~~ a W8x31 column, but it is reasonably indicative of the behavior of all other size columns.

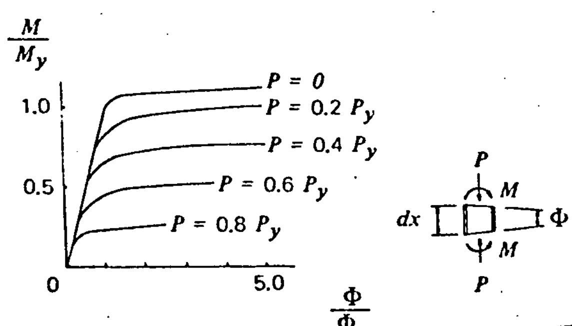


FIG. ~~3.10~~ 3.11 M-P-Φ DIAGRAM FOR ~~8WF31~~ W8x31 (STRONG AXIS BENDING) WITH RESIDUAL STRESS

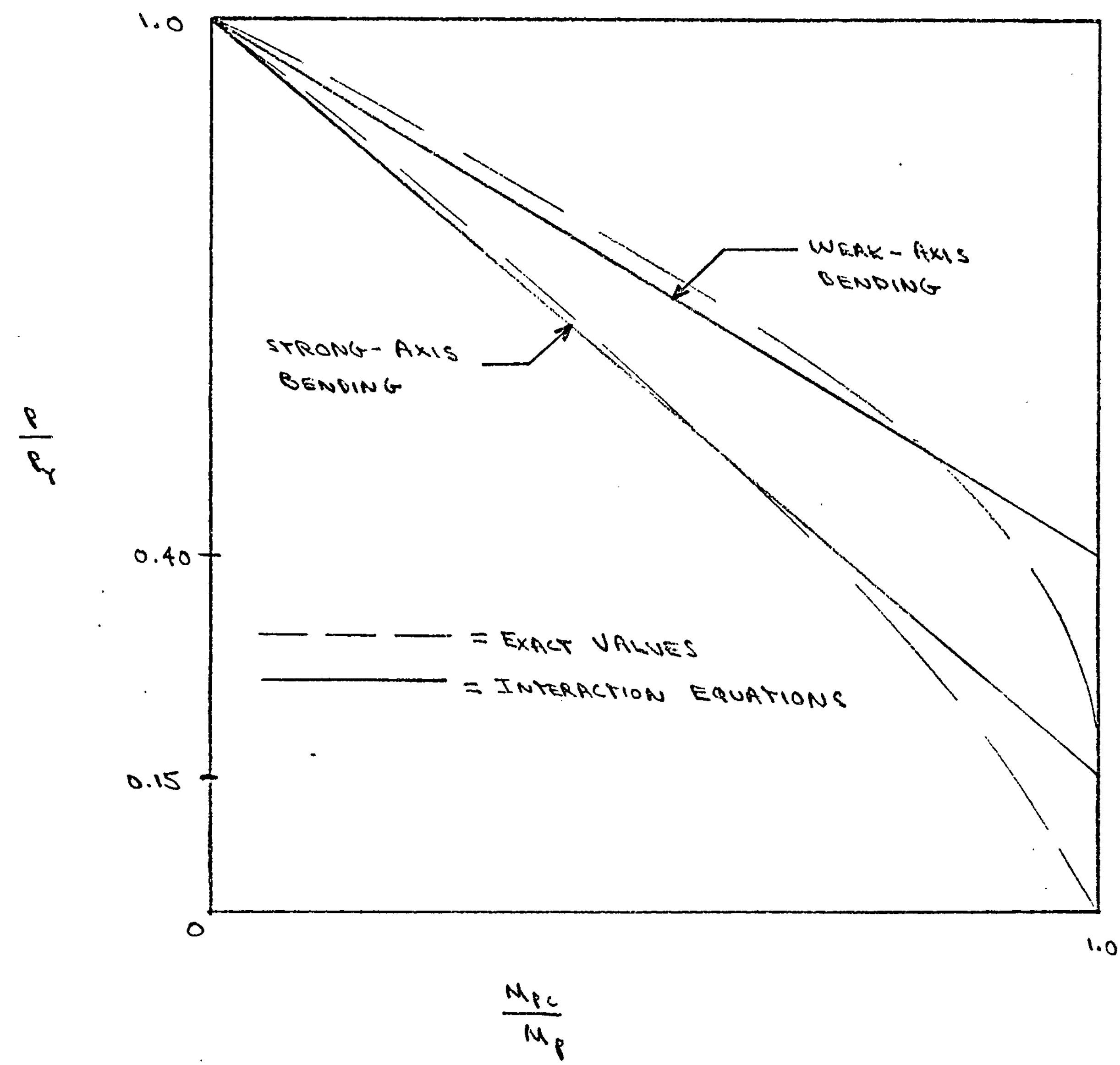


Fig. 3.10 INTERACTION OF AXIAL FORCE AND MOMENT FOR SHORT BEAM-COLUMNS

Using the M-P-Φ curves it is possible, by numerical integration, to represent the ultimate strength of beam-columns by a series of "end moment—end rotation", M-θ curves. The end moments play an important role in influencing the behavior of the beam-column. Several important cases for strong axis bending are illustrated in Fig ~~3.11~~ for beam-columns with $h/r = 30$ and 3.12
$P/P_y = 0.6$. The charts of Design Aid II show M-θ curves for two end moment conditions and values of P/P_y from 0.3 to 0.9 for beam-columns bent about the strong axis.

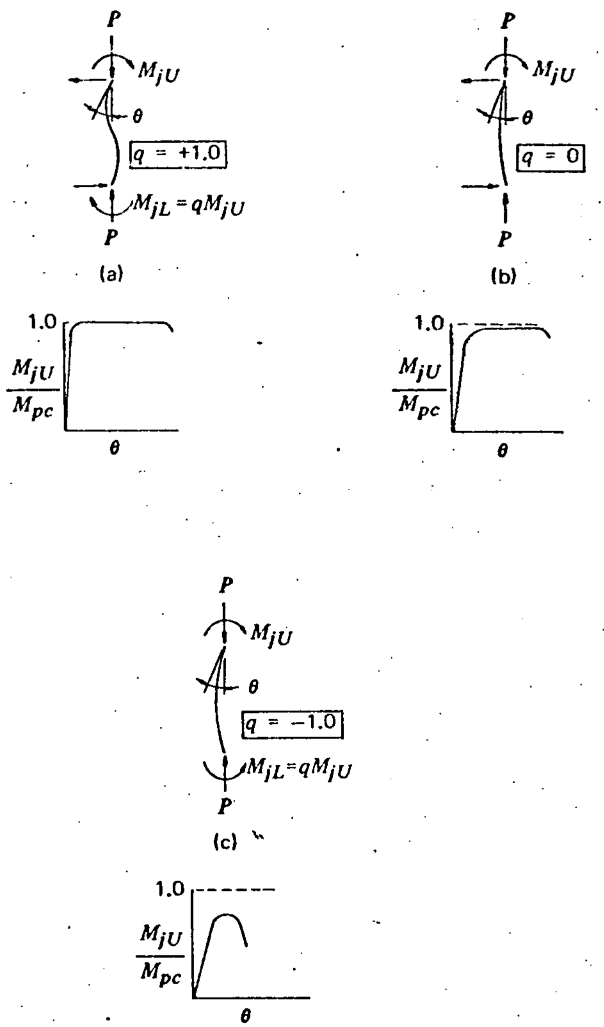


FIG ~~3.11~~ MOMENT-ROTATION CURVES 3.12
(STRONG AXIS BENDING)

In Fig ~~3.11a~~ a beam-column is bent in double curvature by end moments of equal magnitude acting in the same direction, $q = +1.0$. This is a favorable configuration in which the plastic hinges form at the ends at a value of M_{pc}, and are maintained through a considerable rotation. 3.12a

In the beam-column of Fig ~~3.11b~~ bending is produced by a moment at one end only, $q = 0$. Even in this case the maximum moment that can be developed at the end is practically M_{pc}. However, study of Design Aid II will show that for greater slenderness ratios and higher ratios of P/P_y there may be a reduction below M_{pc}. 3.12b

The Design Aid charts for $q = 0$ may be used for the case of $q = +1.0$ by using an equivalent slenderness ratio equal to one-half of the actual. For A36 steel columns bent in double curvature it is only for $P/P_y > 0.9$ and $h/r > 40$ that there is an appreciable reduction below M_{pc}.

In Fig ~~3.11c~~ the beam-column is bent in single curvature by equal end moments, $q = -1.0$. The plastic hinge does not occur at the ends, the end moments never reach the value of M_{pc}, rotation capacity is reduced, and unloading occurs after a small rotation. 3.12c

The charts of Design Aid II may be used for design by assuming a column size, calculating P/P_y and h/r, entering the appropriate chart for P/P_y and q, and reading the maximum value of M/M_{pc}. The latter value multiplied by M_{pc} must equal or exceed the given external moment M for the design to be satisfactory. These charts are most useful when making sub-assemblage checks of the design where joint rotations are of concern. The end points on the moment-rotation curves represent the development of local buckling.

For steel other than A36 the same curves may be used by calculating an equivalent slenderness ratio as follows:

$$\left(\frac{h}{r_x}\right)_{\text{equiv.}} = \left(\frac{h}{r_x}\right)_{\text{actual}} \sqrt{\frac{F_y}{36}} \qquad \cancel{(3.6)} \quad 3.11$$

and modifying the rotation obtained by

$$\Theta = \Theta_{\text{chart}} \sqrt{\frac{F_y}{36}} \qquad \cancel{(3.7)} \quad 3.12$$

Curves for other values of q are available [1] but those given in Design Aid II are usually sufficient for design purposes.

A second method of designing beam-columns uses strong axis interaction curves obtained by plotting the maximum moments from the moment rotation curves of Design Aid II for various values of P/P_y and h/r. The right hand charts of Design Aid III were obtained in this way. Since the in-plane bending strength of ~~W~~ W sections is insensitive to the actual cross-section dimensions, diagrams such as these will suffice for all members.

An additional requirement for beam-columns is

$$P \leq .75\ A\ F_y$$

for the same reasons as given in Art. 3.4 for axially loaded columns.

3.5b STABILITY OF BEAM COLUMNS

If a beam-column has significantly different section properties for the major and minor axes, and if the external moments are applied about the major axis, unbraced beam-columns may experience lateral-torsional buckling before the in-plane bending capacity is reached. The rotation capacity will also be impaired. A conservative estimate of the lateral-torsional buckling strength of beam-columns bent about the major axis by end moments may be made by the following interaction equation.

$$\frac{P}{P_{cr}} + \frac{C_m\ M}{\left(1 - \frac{P}{P_e}\right) M_m} \leq 1.0 \qquad (3.13)$$

where:

P = applied factored axial load

P_{cr} = maximum strength of an axially loaded compression member, Eq. 3.5

M = numerically larger end moment

P_e = elastic buckling load = $\frac{23}{12} A\ F'_e$

where $$F'_e = \frac{12\pi^2 E}{23 \left(K \frac{\ell}{r}\right)^2}$$

$C_m = 0.6 - 0.4\,q$, but $C_m \geq 0.4$

M_m = maximum column moment without axial load

For columns braced in the weak direction (Art. 3.4), M_m is given below as

$$M_m = M_p \tag{3.15}$$

For columns unbraced in thesweak direction, M_m is given below as

$$M_m = \left[1.07 - \frac{\ell/r_y \sqrt{F_y}}{3160}\right] M_p \leq M_p \tag{3.16}$$

In Eq. 3.10 for F'_e, K = 1 can be used provided the secondary PΔ moments are included (Art. 3.6). The appropriate ℓ/r slenderness ratios to be used are given in Table 3.3, depending on whether the column is braced or unbraced, and if bending is about the weak or strong axis.

Design Aid III includes three pairs of charts that give the moment capacity of A36 steel ~~W~~ Wide-flange beam-columns bent about the major axis with a constant end moment ratio q. Charts are provided for double curvature bending ($q = +1.0$), one end pinned ($q = 0$), and single curvature bending ($q = -1.0$).

The first chart of each pair is based on the lateral-torsional buckling (*LTB* for brevity) moment capacity derived from Eq. ~~3.8~~ 3.13 for specified values of h/r_y.

The *LTB* charts assume that the beam-column is braced about both axes only at its ends and that $r_x/r_y = 1.7$, which is a common ratio for W columns of width equal to depth; other light sections have higher values of r_x/r_y. These charts give slightly conservative values of Eq. ~~3.8~~ 3.13 for ~~W~~ Wide-flange columns with $r_x/r_y > 1.7$. The intercepts of the h/r_y curves on the load (P/Py) axis are the ratios P_{oy}/P_y where P_{oy} is the minor axis buckling load from Eq. ~~3.4~~ 3.5. Hence, the *LTB* charts automatically provide a check for minor axis column buckling due to concentric load.

The second chart is based on the maximum in-plane bending moment capacity determined from the peaks of the M-θ curves for specified values of h/r_x in Design Aid II.

The horizontal coordinate axis of the interaction charts indicates the beam-column moment capacity in the form M/M_{pc}. The reduced plastic moment M_{pc} from ~~Eq 3.3 is an~~ Eqs. 3.9 and 3.10 is an upper bound on the moment capacity of ~~the~~ Wide-flange beam-columns bent about the major axis. Note that the axial load ratio P/P_y is used both to enter the interaction charts and to determine M_{pc}.

Design Aids II and III may be used for steels with other values of F_y by entering the curves with an equivalent slenderness ratio from Eq ~~3.6~~ 3.11 and by modifying the end rotation θ using Eq ~~3.7~~ 3.12.

The M-θ curves in Design Aid II are based on in-plane behavior only. If the beam-column moment exceeds the lateral-torsional buckling moment capacity from Design Aid III, lateral bracing must be provided to ensure in-plane behavior. If the beam-column is unbraced between its ends, the M-θ curve is valid only for moments less than the lateral-torsional buckling moment. For an unbraced beam-column in *single curvature* bending ($q = -1.0$), lateral-torsional buckling *always* limits the maximum moment capacity to a value below the peak of the M-θ curve. In the more usual case of double curvature bending ($q = +1.0$), the maximum in-plane moment capacity of an unbraced beam-column can frequently be attained without lateral-torsional buckling, depending on the minor axis slenderness h/r_y and the axial load ratio P/P_y.

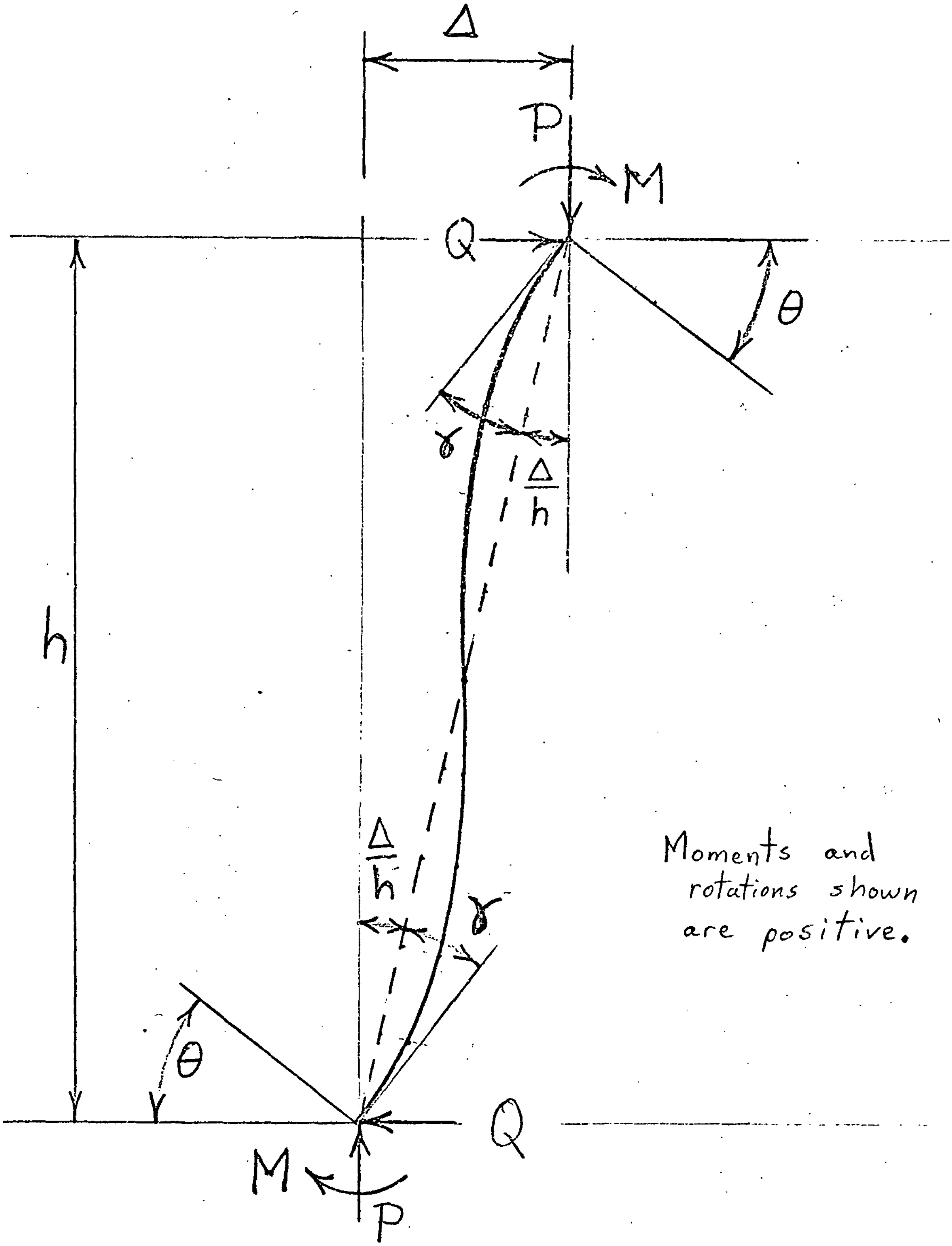


Fig. 3.13: Beam-Column with Sway

The behavior of beam-columns illustrated by the M-θ curves of Design Aid II will not develop if a local buckle of the flange or web occurs. To prevent an early occurrence of local buckling the width-thickness ratio of the component parts must be limited to certain values as shown in Table 3.3.

TABLE ~~3.3~~ 3.4

Specified Minimum Yield Point, F_y	Flange b/t	Web d/w
36 ksi	17.4	70–100 P/P_y but need not be less than 43
50 ksi	14.8	60–85 P/P_y but need not be less than 36

3.6 BEAM-COLUMNS WITH SWAY

In the clear length between joints, the moment-rotation behavior of a column in an unbraced frame is indentical to that in a braced frame. However, in the unbraced frame, some of the column's moment resisting capacity is used up in resisting the additional end moments caused by the PΔ effect ("P-delta" effect) described in Art. 2.2. The statics of such a column may be developed using the free body diagram in Fig. 3.13. The column of height h has its top displaced laterally an amount Δ, giving a chord rotation Δ/h. The rotation of the joint θ is the same as the end rotation of the beams at the joint. The end moments M on the column cause end rotations γ with respect to the column chord. The rotations γ are identical to the rotations θ of the column in Fig. 3.13a, but a different symbol is introduced to differentiate from the beam end rotations.

INSERT FIG. 3.13

By taking moments about one end of the column and solving for Q,

$$Q = -2\frac{M}{h} - \frac{P\Delta}{h} \tag{3.17}$$

Examination of the rotation angles gives

$$\frac{\Delta}{h} = \theta - \gamma \tag{3.18}$$

It should be noted that the typical end moments for sway to the right under the loads P and Q shown would be counterclockwise, opposite from the direction shown in Fig. 3.13. Solving Eq. 3.17 for M gives a negative value agreeing with this intuitive result.

$$M = -\frac{1}{2}\,Qh - \frac{1}{2}\,P\Delta \tag{3.19}$$

In turn the column end rotation γ will receive a negative sign in Eg. 3.18 resulting in a numerical value of Δ/h greater than θ.

Equations 3.17, 3.18, and 3.19 will be used in Chapter 4.

CHAPTER 4

ADDITIONAL PLASTIC DESIGN TECHNIQUES FOR UNBRACED FRAMES

4.1 EQUILIBRIUM OF FRAME IN ITS DISPLACED POSITION

Certain equilibrium relationships are satisfied by any unbraced frame in its displaced position. Equations expressing this equilibrium are useful in both the preliminary design and the final analysis of the frame.

One such equilibrium equation is based on the free body diagram of Fig. 4.1 showing the several columns in a story which is subjected to a horizontal shear ΣH and gravity load ΣP. The values of ΣH and ΣP are computed from the loads acting on all the stories above the one shown in Fig. 4.1, a story which has a sway Δ and a height h. The resultant horizontal shear and total gravity load acting together in the deflected position cause an overturning moment which must be resisted by the sum of the column end moments, ΣM_c. Without knowing the individual end moments, their total sum can be determined from

$$\Sigma M_c = -[(\Sigma H)h + (\Sigma P)\,\Delta] \qquad (4.1)$$

In any floor level the beams can receive column moments from the columns above and below. The total of the column moments in the stories above and below the floor level may be determined from two calculations of Eq. 4.1. To determine that portion of the column moments which act at the top and bottom of each story requires either an elastic analysis or an accurate estimate. For design estimates it is suitable for most stories to assume that half the total moments are at the top and bottom of each set of columns. This estimate is equivalent to assuming an inflection point at midheight. Then the sum ΣM_g of the end moments on all beams in a level is

$$\Sigma M_g = -\tfrac{1}{2}\,[(\Sigma M_c)_{n-1} + (\Sigma M_c)_n] \qquad (4.2)$$

in which n-1 refers to column moments in the story above and n to those in the story below the beams at floor level n, as shown in Fig. 4.2

The drift Δ which affects ΣM_c in both Eq. 4.1 and Eq. 4.2 is unknown at the time of preliminary analysis. It can be estimated so that trial member sizes can be selected and then revised if later deflection checks show this to be necessary.

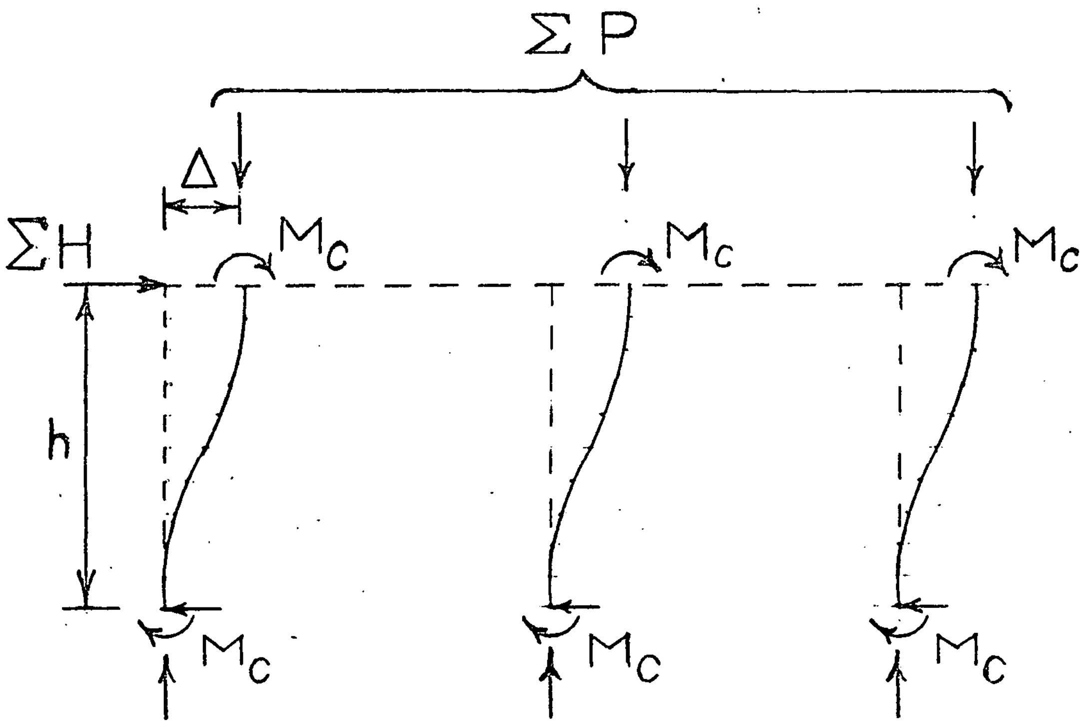


Fig. 4.1 Horizontal Shear Equilibrium in a Story of an Unbraced Frame

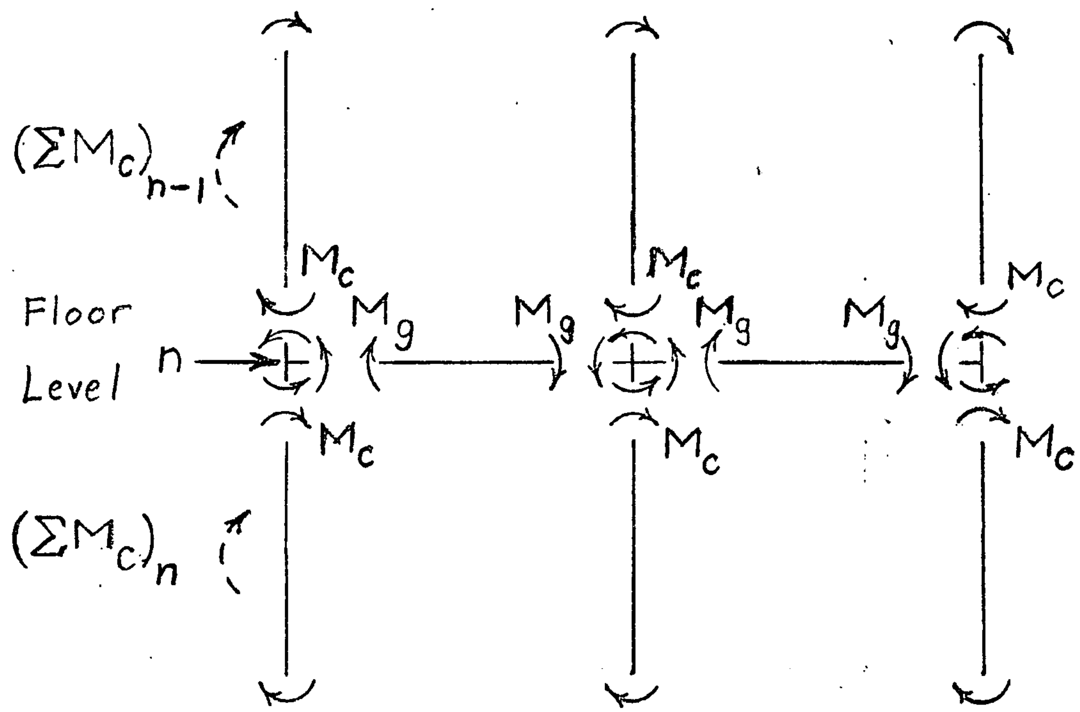


Fig. 4.2 Free Body Diagram of Moments at Floor Level n

Calculation of Eq. 4.1 once for each story and Eq. 4.2 once for each floor level will give moments useful in the preliminary design of beams and in the moment balancing procedure leading to column design moments.

4.2 MOMENTS IN BEAMS

Moments in beams are key quantities in two situations connected with design of multistory frames: (1) design, and (2) review after design. In either of these two situations, three cases must be treated. Beams must be adequate either for gravity load alone or to resist gravity load in combination with lateral load from either direction in the plane of the frame.

Structural design is based upon the maximum moment caused by the loading which occurs at any point within the clear span of the beam. This moment cannot (except with strain-hardening) exceed the plastic hinge moment M_p. For review of the structure after either a trial design or a final design, the values of beam end moments calculated at the theoretical intersection points of beams and columns are needed.

For both these situations, formulas can be generated from the statics of a beam transversely loaded with uniform load and subjected to end moments. For any beam, the complete moment diagram will be known if the transverse load is known plus the moments at any two discrete points. Boundary conditions help to define some of these moments. This treatment will cover the cases of boundary conditions most occurring in multistory frames: beams rigidly connected at both ends, and beams rigidly connected at one end and having a simple connection at the other end.

Design Formulas

Formulas for design moments in beams may be presented and used in either equation form or in the form of graphs. One form of such graph is given in Fig. 4.3. The equations of the curves presented are given in Table 4.1.

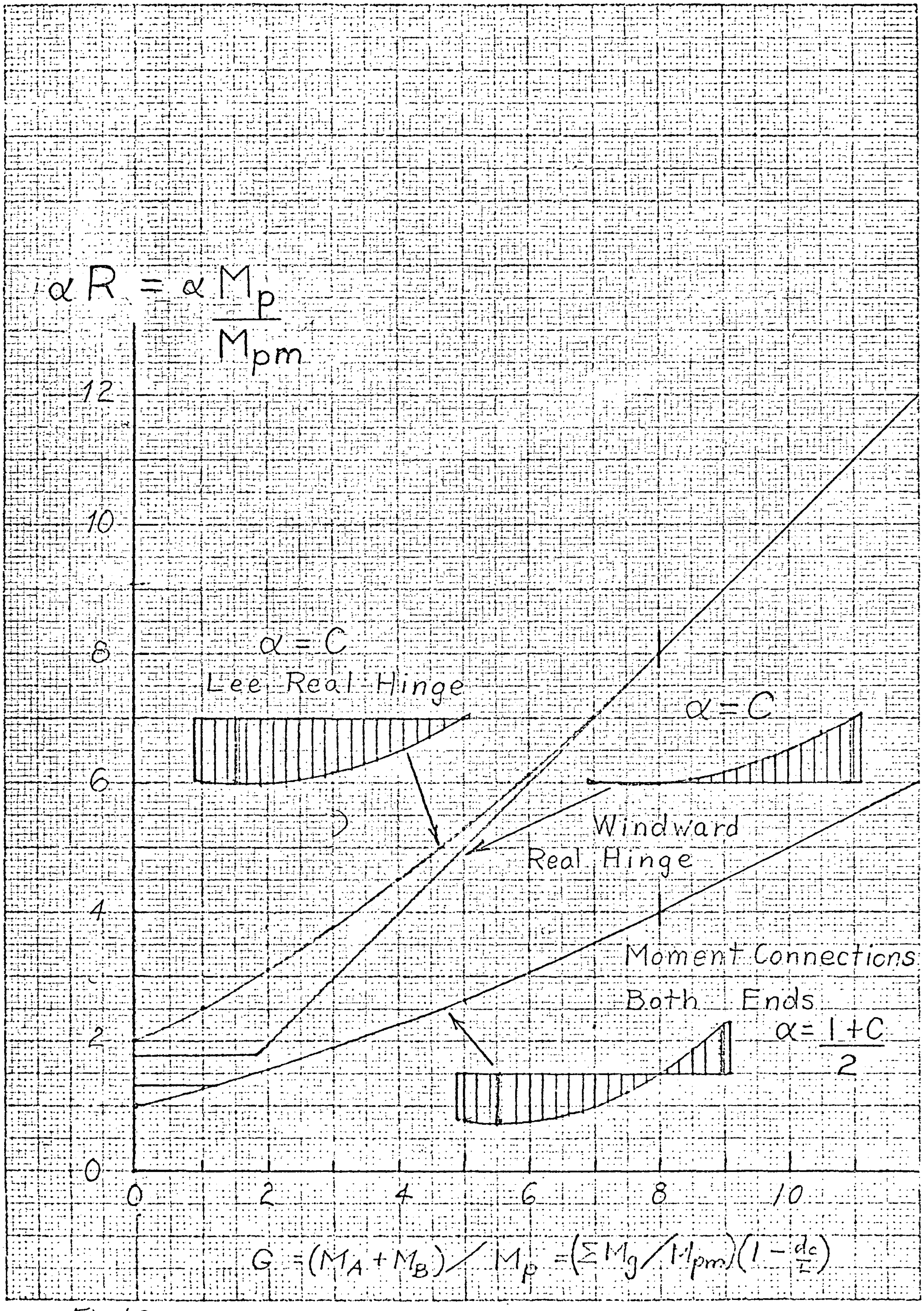

$\alpha R = \alpha \frac{M_p}{M_{pm}}$
12
10
8
6
4
2
0
$\alpha = C$
Lee Real Hinge
$\alpha = C$
Windward
Real Hinge
Moment Connections
Both Ends
$\alpha = \frac{1+C}{2}$
0
2
4
6
8
10
$G = (M_A + M_B) / M_p = (\Sigma M_g / M_{pm})(1 - \frac{d_c}{L})$

Fig 4,3

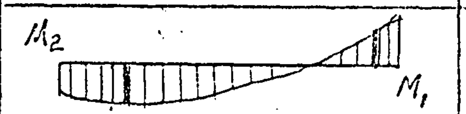
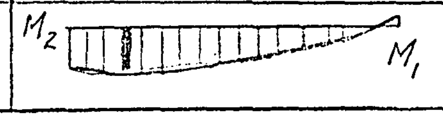
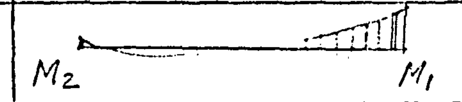
BEAM MOMENTS		Two Moment Connections		Lee Simple Connection		Windward Simple Connection	
							
Design Parameters		Gravity Capacity Required Wind Capacity Required			$M_{pm} = \frac{1}{16} F_2\, w(L-d_c)^2$ $G = \frac{\Sigma M_g}{M_{pm}}\left(1-\frac{d_c}{L}\right) = \frac{(M_1+M_2)}{M_{pm}}\left(1-\frac{d_c}{L}\right)$		
DESIGN EQUATIONS		G (given)	R (required)	G (given)	R (required)	G (given)	R (required)
$R = \frac{M_p}{M_{pm}}$	Gravity Controls	$0 \le G \le 1.15$	$R = \frac{2}{1+C}\frac{F_1}{F_2}$			$0 \le G \le 1.80$	$R = \frac{16}{11.66}\frac{F_1}{F_2} = \frac{1.80}{C}$
$0 \le C \le 1$	Combined Mechanism	$1.15 \le G \le 8$	$R = \frac{2}{1+C}\left(1+\frac{G}{8}\right)^2$	$0 \le G \le 8$	$R = \frac{2}{C}\left(1+\frac{G}{8}\right)^2$		
Default $(C=1)$	Wind Controls	$8 \le G$	$R = \frac{G}{1+C}$	$8 \le G$	$R = \frac{G}{C}$	$1.80 \le G$	$R = \frac{G}{C}$
Review Parameters		Clear Span Parameter $D = \frac{d_c}{L} \Big/ \left(1-\frac{d_c}{L}\right)$					
REVIEW EQUATIONS		R (given)	G (required)	R (given)	G (required)	R (given)	G (required)
		$1.31 \le R \le 4$	$G = 8\left[\sqrt{R} - 1.0\right]$	$2 \le R \le 8$	$G = 8\left[\sqrt{\frac{R}{2}} - 1.0\right]$	$R_{min} = 1.80$	$0 \le G \le 1.80$
		$4 \le R$	$G = 2R$	$8 \le R$	$G = R$	$1.80 \le R$	$G = R$
Beam Moments Formulated at Column Centerlines	Lee ward End	$\frac{M_1}{M_{pm}} = R + (4+\frac{1}{2}G)D$		$\frac{M_1}{M_{pm}} = (4+\frac{1}{2}G)D$ *		$\frac{M_1}{M_{pm}} = R + (4+\frac{1}{2}G)D$	
	Windward End	$\frac{M_2}{M_{pm}} = G - R - (4-\frac{1}{2}G)D$		$\frac{M_2}{M_{pm}} = G - (4-\frac{1}{2}G)D$		$\frac{M_2}{M_{pm}} = -(4-\frac{1}{2}G)D$ *	
	Sum	$\frac{\Sigma M_g}{M_{pm}} = G + GD$					
		* NOTE: Very small moment at end with simple connection.					

Table 4.1

Design must begin with a calculation of the gravity capacity required and the wind (or other lateral load) capacity required. Gravity capacity is reflected in a parameter M_{pm} which is used to non-dimensionalize many of the moments.

$$M_{pm} = \frac{1}{16} F_2 \, w(L - d_c)^2 \tag{4.3}$$

where F_2 = load factor for combined loading (1.30)

w = uniform load

L = center-to-center beam span

d_c = average column depth at ends of span

Wind capacity required is reflected in a parameter G which relates the wind moments required to the gravity parameter M_{pm}.

$$G = \frac{\Sigma M_g}{M_{pm}} \left(1. - \frac{d_c}{L}\right) = \frac{(M_1 + M_2)}{M_{pm}} \left(1. - \frac{d_c}{L}\right) \tag{4.4}$$

where M_g = sum of wind moments beam must resist

M_1 = lee end moment (formulated at center of joint)

M_2 = windward end moment (formulated at center of joint)

A further optional parameter called a positive moment factor C may be used to provide maximum sagging moments less than M_p in order to control deflections artificially. By using a C value less than one, the maximum positive (sagging) moments will be limited to CM_p when the structure is in equilibrium with the design load.

The final result of a design calculation is the determination of the required plastic moment value for the beam. This is contained in the non-dimensional parameter R which is the ratio of M_p/M_{pm}.

Failure modes controlling design are found to fall in three domains based on the relationship of wind moments to gravity moments as measured by the parameter G.

(1) Small wind moments -- gravity load controls design.

(2) Moderate wind moments -- combined failure. Beams must be increased in size above those required to carry gravity load alone.

(3) Large wind moments -- a panel mechanism forms with plastic hinges at the ends of beams. Plastic moments are dominated by wind moments even though gravity moments are still present.

For a beam with two moment connections the equations for the three domains are:

(1) $\quad R = \dfrac{2}{(1. + C)} \dfrac{F_1}{F_2} \qquad 0 \leq G \leq 1.15$

(2) $\quad R = \dfrac{2}{(1. + C)} (1. + \dfrac{G}{8})^2 \qquad 1.15 \leq G \leq 8$

(3) $\quad R = \dfrac{\cancel{2} \cdot G}{(1. + C)} \qquad 8 \leq G$

where F_1 = load factor for gravity loading (1.70)

When these equations are evaluated for C = 1, the following observations can be made. If the wind moments required (as indicated by the G factor) are less than about 1.15 M_{pm} the plastic moment capacity required is limited to a minimum of F_1/F_2 or about 1.31 M_{pm}. If the wind moments are greater so that a combined mechanism forms, the required M_p turns out to be something greater than half the wind moment. If wind moments become larger than 8 M_{pm}, the plastic hinge moment required becomes exactly half the wind moment carried by the beam.

For a beam with a simple connection at the lee end there are equations for only two of the domains. These are:

(2) $\quad R = \dfrac{2}{C} (1. + \dfrac{G}{8})^2 \qquad 0 \leq G \leq 8$

(3) $\quad R = \dfrac{G}{C} \qquad 8 \leq G$

These equations are evaluated for C = 1. The required M_p is greater than the wind moment when the wind moments are less than 8 M_{pm}. If wind moments are larger than 8 M_{pm} the plastic hinge moment required equals the wind moment carried by the beam.

For a beam with a simple connection at the windward end there is no combined mechanism and either gravity alone controls or wind controls. The equations are:

$$(1) \quad R = \frac{16}{11.66} \frac{F_1}{F_2} = \frac{1.80}{C} \qquad 0 \leq G \leq 1.80$$

$$(2) \quad R = \frac{G}{C} \qquad 1.80 \leq G$$

These equations imply that there is a minimum required plastic hinge moment controlled by gravity when wind moments are small and thereafter the plastic hinge moment required is exactly equal to the wind moment carried by the beam.

The following example will illustrate the use of the design formulas.

<u>Example 4.1</u>

Determine the required plastic moment of beams needed for a story of a frame having the following conditions. The story heights are 10 feet. There are two beam spans, L of 15 feet. The uniform load w is 0.6. Column depths are assumed to be ½ ft. and the positive moment factor C is assumed to be 1.0. Assume that the calculations using equation 4.1 have determined sums of column moments as follows:

$$\Sigma M_c = -60 \text{ k-ft, above level}$$

$$\Sigma M_c = -72 \text{ k-ft, below level}$$

Inserting these values into equation 4.2 gives the sum of the end moments on the beams as follows:

$$\Sigma M_g = -\tfrac{1}{2}\,(-60 - 72) = 66^{\text{k-ft}}$$

Since both beams are the same span assume 33 k-ft on each beam. Using equation 4.3

$$M_{pm} = \frac{1}{16} F_2 w(L-dc)^2 = \frac{1}{16} (1.3)\ (0.6)\ (14.5)^2 = 10.25 \text{ k-ft}$$

The wind capacity parameter G is determined from equation 4.2.2.

$$G = \frac{M_g}{M_{pm}} (1.0 \frac{dc}{L}) = \frac{33}{10.25} (1.0 - \frac{0.5}{15}) = 3.11.$$

Since $1.15 \leq G \leq 8$ use

$$R = \frac{2}{1. + C} (1. + \frac{G}{8})^2 = \frac{2}{2} (1. + \frac{3.11}{8})^2 = 1.93$$

Therefore the M_p required is:

$$M_p = 1.93\ M_{pm} = 1.93\ (10.25) = 19.76 \text{ k-ft}$$

A suitable member would be selected with an M_p value greater than 19.76. Assume that $M_p = 20$ would be tried.

Review Formulas

For review of a design the object is to determine the resisting moment capacity measured by parameter G for a given plastic moment capacity measured by R. These parameters have the same definition as previously and there is one additional parameter used, a clear span parameter D.

$$D = \frac{d_c}{L} / (1. - \frac{d_c}{L}) \qquad (4.5)$$

Limiting moment cases in analysis fall in two domains based on the dimensionless plastic moment parameter R. (1) Small plastic moment -- a plastic hinge forms within the clear span. (2) Large plastic moment capacity -- plastic hinges form only at ends of members.

Beam with Two Moment Connections

For a beam with two moment connections the equations for the two domains are:

(1) $G = 8[\sqrt{R} - 1.0]$ $1.31 \leq R \leq 4$

(2) $G = 2R$ $4 \leq R$

Once the parameter G is known the limiting end moments can be formulated at the column center lines.

Beam Moments Formulated at Column Centerlines

Leeward End

$$\frac{M_1}{M_{pm}} = R + (4 + \tfrac{1}{2} G)D$$

Windward End

$$\frac{M_1}{M_{pm}} = G - R - (4 - \tfrac{1}{2} G)D$$

The sum of the end moments is given by:

$$\frac{\Sigma M_g}{M_{pm}} = G + GD$$

Beam with Simple Connection at the Lee End

For a beam with a simple connection at the lee end, the equations for the wind moment capacity are as follows:

(1) $G = 8[\sqrt{\frac{R}{2}} - 1.0]$ $2 \leq R \leq 8$

(2) $G = R$ $8 \leq R$

Beam Moments Formulated at Column Centerlines

Leeward End

$$\frac{M_1}{M_{pm}} = (4 + \tfrac{1}{2}\ G)D$$

Windward End

$$\frac{M_2}{M_{pm}} = G - (4 - \tfrac{1}{2}\ G)D$$

When values for G and D are substituted into these expressions M_1 will be found to be a very small quantity. It is the small moment caused by the beam end shear at the simple connection being applied eccentrically to the column center line.

Beam With Simple Connection at the Windward End

For a beam with a simple connection at the windward end, the expressions for wind moment connections are as follows:

(1) $0 \leq G \leq 1.80$ $\qquad R_{min} = 1.80$

(2) $G = R$ $\qquad 1.80 \leq R$

Beam Moment Formulated at Column Centerlines

Leeward End

$$\frac{M_1}{M_{pm}} = R + (4 + \tfrac{1}{2}\ G)D$$

Windward End

$$\frac{M_2}{M_{pm}} = -\ (4 - \tfrac{1}{2}\ G)D$$

As in the previous case the moment M_2 is only the small moment caused by the eccentricity of the beam end shear applied at the column face.

The review equations for beam end moments are useful in making analysis of a frame by the moment balancing procedure or analyzing the strength and deflection of sub-assemblages by the sway sub-assemblage method.

The use of the review equations for beam end moments will be illustrated by the following example which prepares data for the moment balancing process.

Example 4.2

Find the end moments on the beams selected in example 4.1 when subjected to the loads of example 4.1.

First determine the plastic moment ratio R, and the clear span parameter D from the given quantities.

$$R = \frac{M_p}{M_{pm}} = \frac{20}{10.25} = 1.951$$

$$D = \frac{0.5}{15} \; / \; (1. - \frac{0.5}{15}) = \frac{1}{29}$$

Since $1.31 \leq R \leq 4$ use

$$G = 8(\sqrt{R} - 1.0) = 8(\sqrt{1.951} - 1.0) = 3.176$$

Once the parameter G is known the end moments M_1 and M_2 can be calculated.

$$\frac{M_1}{M_{pm}} = R + (4 + \tfrac{1}{2}G) \; D = 1.951 + (4 + 1.588) \frac{1}{29} = \overset{2.144}{\cancel{1.142}}$$

$$\frac{M_2}{M_{pm}} = G - R - (4 - \tfrac{1}{2}G) \; D = 3.176 - 1.951 - (4 - 1.588) \frac{1}{29} = 1.142$$

These dimensionless values of M_1 and M_2, may now be converted into their values in k-ft by multiplying by the non-dimensional parameter M_{pm}.

$$M_1 = 10.25 \; (2.144) = 21.976$$

$$M_2 = 10.25 \; (1.142) = 11.70$$

$$\Sigma M_g = 21.98 + 11.70 = 33.68$$

Note that this value is slightly larger than the 33 k-ft which was needed. This is a result of selecting a member larger than the minimum size needed, a typical situation. In the fully loaded structure the member would continue to have plastic hinges at the lee end but the moment would be less at the windward end. Assume rounded figures of 22 and 11 k-ft. These values will be used for girder moments in an illustration of the moment balancing process.

4.3 MOMENT BALANCING

For any design of highly indeterminate framed structures it is desirable to have a preliminary analysis method for determining bending moments before any member sizes are selected. In allowable stress design, engineers have used methods such as the portal and cantilever methods for many years. For plastic design, a method similar to the portal method and called "moment balancing" has been developed. It will be discussed here.

Concepts of Moment Balancing

Both moment balancing and the portal method provide a set of moments which is in equilibrium with the horizontal loads but which ignores the compatibility of deflections. Both methods achieve an equilibrium solution by making assumptions of bending moment and shear at sufficient points to reduce the structure to determinacy. The remaining forces in the structure are determined by statics considering equilibrium at the joints and within each story.

Figure 4.4 gives a schematic comparison between the assumptions of the portal method and the moment balancing method.

(1) Both methods initially assume inflection points (that is: $M = 0$) at midheight of the columns.

(2) In the portal method, the analysis of the columns is completed by applying to each of the columns in a story an empirical percentage of the total horizontal shear applied to the story. From the shears the end moments are calculated.

In the moment balancing method, the sum of all column end moments in a story is determined from the story shears and then an empirical percentage of the sum is applied to each end of each column.

The two concepts of handling column equilibrium may actually be considered to be identical--only the handling of the arithmetic differs. In some applications of the portal method, the shear resisted by each column is assumed to be proportional to the aisle width supported by the column. In other applications of the portal method each exterior column is assumed to carry one-half the shear carried by each interior column. Undoubtedly both distributions are averages based on experience with exact analyses of some frames.

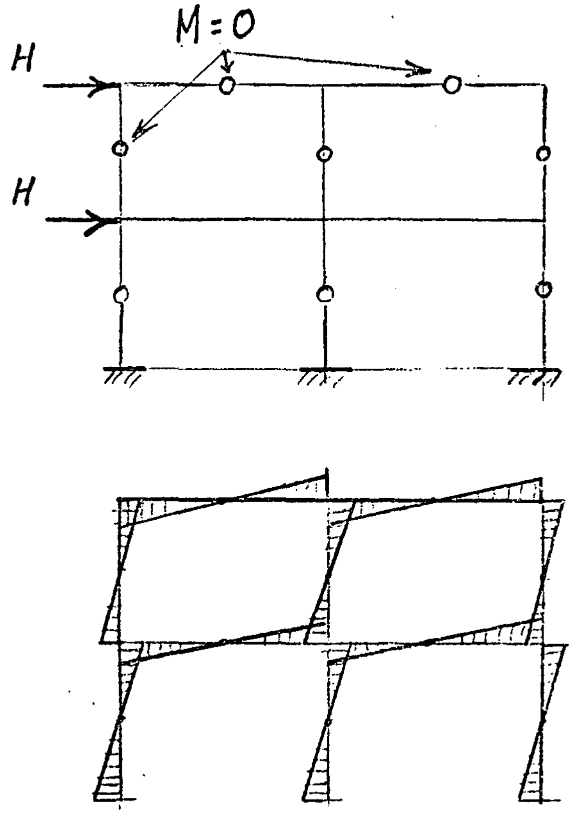
Because of the adjustments later needed to satisfy equilibrium at the joints, it is more convenient to work with column end moments in the moment balancing method. The typical procedure in examples presented in available literature is to assign equal end moments to each end of each column in a story initially. The distribution between columns is later adjusted to suit the end moments on beams. Undoubtedly it would be equally appropriate to assign initial column end moments in the same proportions as the shears assumed by the portal method.

The third basis for comparison of the two methods is the treatment of moments in the beams.

(3) In the portal method, inflection points are assumed at the mid point of each beam.

In moment balancing, limiting moments depending on the formation of plastic hinges in the beams are determined.

Since inflection points are assumed at the middle of each beam in the portal method, this immediately assigns two equal end moments for each beam.



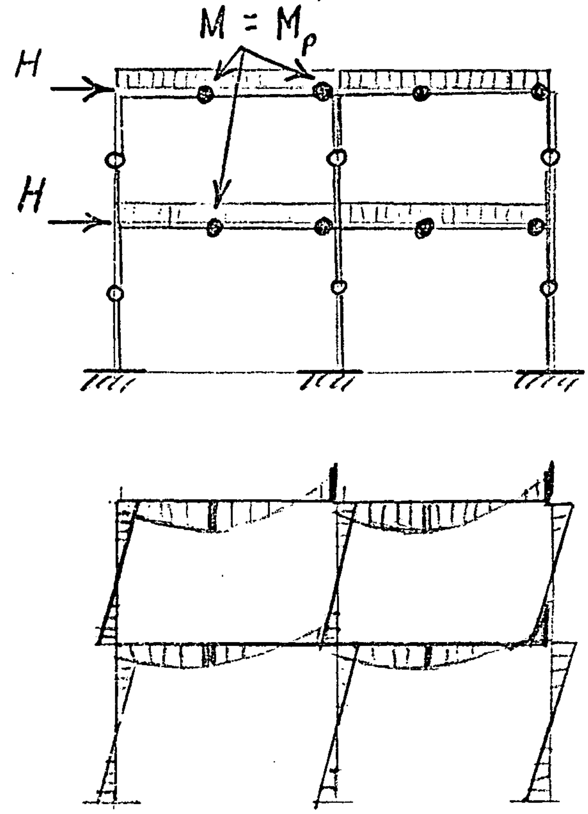
(a) Portal Method

Assumptions:

(1) $M=0$ at Mid-height of columns

(2) Horizontal Shear within a story distributed empirically between columns.

(3) $M=0$ at Mid-span of beams.



(b) Moment Balancing

Assumptions:

(1) $M=0$ at Mid-height of Columns

(2) Moments within a story due to Horizontal Shear distributed empirically between columns.

(3) $M = M_p$ in beams

Fig. 4.4 Assumptions of Portal Method and Moment Balancing Method.

The magnitude of the beam moments is determined by equilibrium calculations at the joints where column end moments have previously been determined. It should be noted that the structure is analyzed for the lateral loads alone in the portal method (Fig. 4.4a). This is feasible because the structure is assumed to remain elastic for all loadings in allowable stress design and the moments from wind loads may be superimposed upon those from floor loads.

In contrast, the plastic design of unbraced frames has two sources of non-linearity which require that the combined moments be considered. Both the P-delta effect and the irregular formation of plastic hinges accompanying the redistribution of moment preclude the use of superposition methods. Instead, beams are selected so that the combined limiting moment diagram (Fig. 3.5d) indicates adequate capacity to resist both the wind and vertical load moments. Then the column moments are brought into balance with the beam moments to satisfy equilibrium at each joint.

Execution of the Moment Balancing Method

It is actually easier to perform a moment balance than it is to describe it. Prior to the moment balance, preliminary design data are obtained by tabulating column loads at each floor due to gravity load. These loads are assumed to be consistent with the tributary floor areas. Horizontal shears for each story are similarly tabulated according to tributary areas acted upon by wind.

The actual moment balance is performed in the following steps:

(1) Determine the sum of the required column end moments in a story. (Art. 4.1)

(2) Determine the sum of the required beam end moments in a story. (Art. 4.1)

(3) Select trial beam size and moment diagram. (Art. 4.2)

(4) Adjust column end moments to be in equilibrium with beam end moments at each joint.

(5) Select column sections. (Art. 4.4)

The first three steps as described in Articles 4.1 and 4.2 would provide independent sets of beam moments and column end moments which would be adequate to resist the applied loads on the structure. However, these moments would be inconsistent with each other at the joints. Adjustments must be made which produce equilibrium moments about each joint without disturbing the equilibrium horizontal loads on the columns and vertical loads on the beams. These adjustments may be made by changing only the column end moments.

Fig. 4.5 shows a sample of the preliminary moment diagrams in some columns and beams of the structure examined in examples 4.1 and 4.2, prior to moment balance. On joint A the column moments sum 22 in a clockwise direction and the beam moment is 11 in a counter clockwise direction, an unbalance of 11. By subtracting about half of this unbalance from the moment in the column above and below the joint, the joint could be brought into balance. Using round figures this could be accomplished by reducing the column moment above the joint by 5 and the column moment below the joint by 6. This would balance joint A, but would leave the total sum of column moments in the story less than needed to resist the horizontal loads. However in any story if one joint is out of balance from the preliminary calculations other joints must also be out of balance. For each reduction in moments of columns necessary at one joint there will be corresponding increases of moments necessary to create balance at other joints.

Examining joint B it will be seen that the girder moments sum to 33 k-ft counter clockwise, and the column end moments sum to 22 k-ft clockwise, an unbalance of 11. If the moment in the column above joint B is increased by 5 and the moment in the column below the joint B is increased

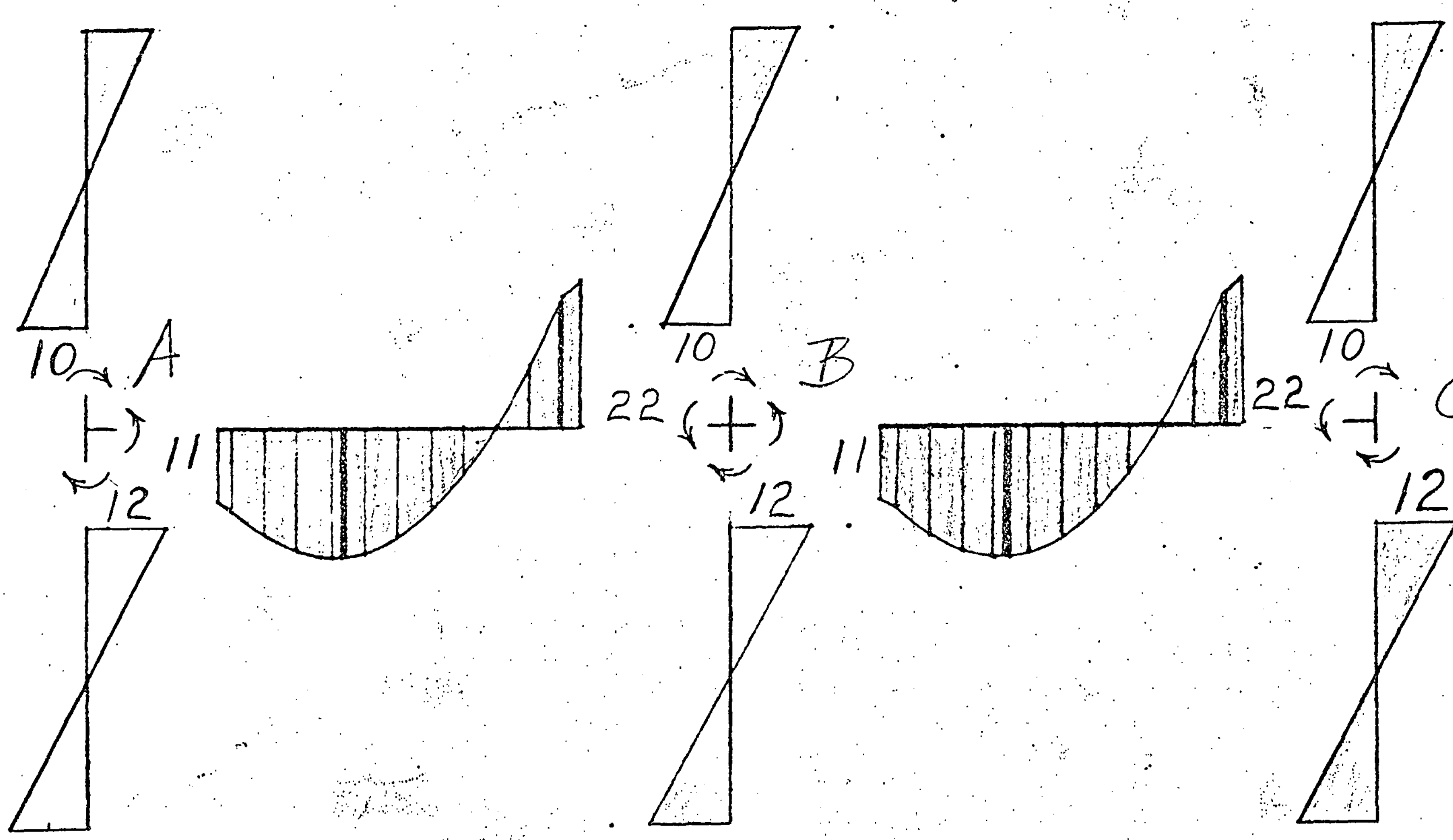


Fig. 4.5 Preliminary Moment Diagrams Prior to Moment Balance

by 6 joint B will be in balance. The sum of changes made will also have resulted in a net zero within each story.

The unbalance at joint C must next be examined. In this case Joint C is seen to be in balance from the original preliminary assignment of moments. This is not the typical case. Usually joint C would usually have been out of balance and a change in column end moments would need to be made. However if the initial column end moments and the initial beam end moments were both calculated properly, the total sum of moments in beams and columns should be exactly correct. Therefore the sum of any corrections made within a story should total out to zero assuring that the equilibrium of horizontal shears was not disturbed. Fig. 4.6 shows the preliminary moment diagrams of the story after moment balancing.

4.4 ANALYSIS FOR DRIFT

One important consideration in the design of unbraced frames is the control of drift. In the preliminary design of the members, it is necessary to estimate the ultimate load drift index, $(\Delta/h)_{ult}$ (see Art. 6.3). This is needed to formulate equilibrium of the frame in its displaced position. After preliminary member sizes have been selected, it is necessary to analyze the frame for drift to check the following:

(1) That sway at working load will cause no damage to non-structural components such as windows, walls, or the exterior facade.

(2) That sway at working load will cause no occupant discomfort.

(3) That at factored load the assumed sway index is less than the actual sway index.

If either of the above three requirements is not met, the stiffness of the frame must be increased. In an unbraced frame this is most effectively accomplished by increasing the girder sizes.

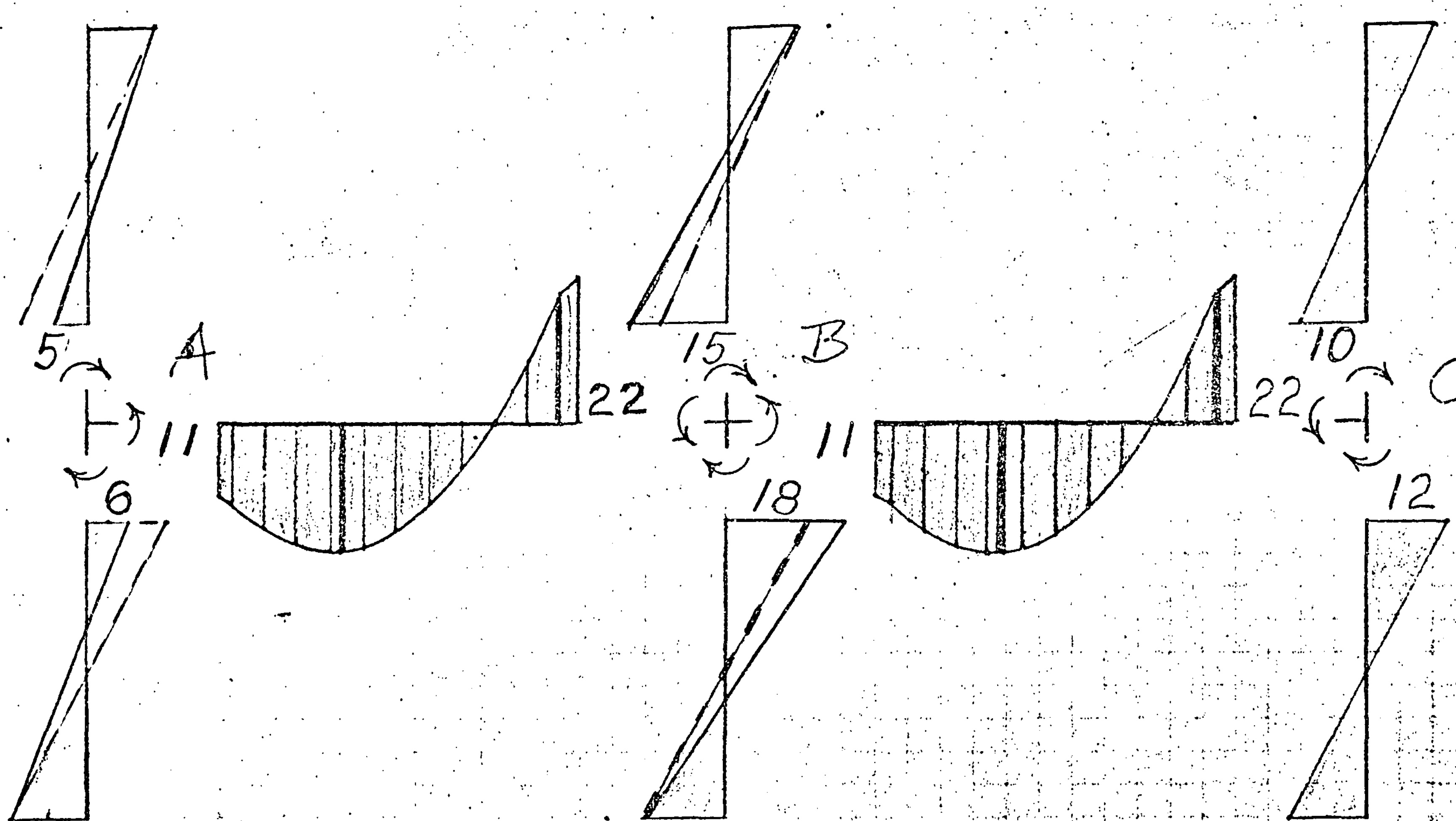


Fig 4.6. Preliminary Moment Diagrams Resulting From Moment Balance

There are many methods available for calculating sway. A detailed discussion of these methods will not be given here. However, to illustrate some of the basic principles involved when calculating sway, the sway subassemblage method will be briefly discussed. A detailed discussion of this method can be found in Refs. 5 to 11.

The sway subassemblage method uses a model called an assemblage to represent a story in a building frame. Fig. 4.7 illustrates a typical story in a frame as represented by an assemblage. The assemblage will consist of the girders and a portion of the columns below the floor level extending down to a row of assumed inflection points. Furthermore, the assemblage is separated into subassemblages, each subassemblage consisting of one of the columns plus the girders framing into the column top. Typical exterior and interior subassemblages are illustrated in Fig. 4.8.

The shear versus drift relationship for the assemblage is determined by a displacement method during an assumed set of joint rotations Θ. Changes in the girder end moments during the rotations Θ can be used to determine other functions such as the column end moments and the drift Δ. The relationship between these functions is calculated as girder moments change from a state of at-rest equilibrium under factored gravity loads ($F = 1.3$) to a final state of combined gravity and lateral load equilibrium. Gravity loads remain at a constant factored level of $F = 1.3$ as the wind loads are increased. The resulting drift Δ versus horizontal shear ΣH relationship for the story can then be plotted, as shown in Fig. 4.9.

It is not necessary to analyze all the stories in a building for drift. For medium sized frames (about 20 stories), it is usually sufficient to analyze a story near the top, middle, and bottom. If each of these stories sways within acceptable limits, it can be assumed that all the stories have an acceptable sway.

CHAPTER 5

Design of Supported Bents for Gravity Loads

~~4~~ 5.1 INTRODUCTION

This is the first of ~~several~~ five chapters which illustrate the plastic design of a braced multistory building. Included in this chapter are a description of the building to be designed and the scope of the design example. This is followed by an explanation of the design of a multistory bent for full gravity loads. The design calculations for the example described here are grouped together in Chapter ~~8~~ 9 for easy reference.

5.2 DESCRIPTION OF BUILDING

The plastic design example concerns a 24-story apartment building. Preliminary structural plans are summarized in Fig. 9.1. The reader is referred to the 1968 AISI Manual entitled "Plastic Design of Braced Multistory Steel Frames" (Ref. ~~9.1~~ 12) in which an identical 24-story apartment building has been designed as a braced frame. The main structural elements are 3-bay bents spaced 24 ft. apart. Tie beams are spandrels between the bents are framed to the columns using simple connections. This structural system is assumed to cause column moments from gravity loads to occur only in the plane of the bents.

Section A-A in Fig. 9.1 indicates the floor framing plan in the supported bents, which includes a 2½ in. lightweight concrete slab on a corrugated steel form supported by open web steel joists. A construction height of 1 ft. - 8 in., with a depth limitation of 14 in. for the girders in the supported bents, combined with an 8 ft. clear ceiling height gives a story height of 9 ft. - 8 in. In the bottom two stories the height is increased to 12 ft. The floor framing plan for the lateral force resistant bents is the same, except that the depth limitation for the girders is 38 in. in the 9 ft. - 8 in. stories. This large depth limitation is necessary in the lateral force resistant bents, especially in the lower stories, because of the large girder moments caused by the wind forces. In most stories these large girders can be concealed as dropped-beams, but will impose some architectural constraints on the floor plan.

The numbering system used to identify members in the design calculations is shown in Fig. 9.1. The column lines are numbered 1 to 4 and the floor levels are numbered from the roof down. The letters A and B designate individual bents, Bents A being supported bents and Bents B being wind-resistant bents.

The lower portion of Fig. ~~8.1~~ 9.1 summarizes the working loads. To simplify the numerical work, the floor loads in the 8 ft. corridor are applied over the full 12 ft. width of the interior bay between column lines 2 and 3.

The intent of this example is to illustrate the application of plastic design concepts to a practical building problem. The framing in Fig. ~~8.1~~ 9.1 is one of several practical structural solutions for this building and should not be regarded as an optimum structural system.

5.2 TRANSFER OF WIND FORCES

The size and shape of the building in Fig. 9.1 suggest that resistance to wind is an important structural consideration. To provide wind resistance either a vertical bracing system, a moment resistant frame, or a combination of both must be used. Vertical bracing is usually the most economical solution when architectural requirements permit its use. When architectural requirements are such that a bracing system would interfere with the functional design of the floor plans, unbraced lateral force resistant bents which have sufficient strength and stiffness to resist the wind forces and accompanying $P\Delta$ effect provide a satisfactory structural framework.

The double solid lines in Fig. 9.1 indicate the location of the force resistant bents. The five lateral force resistant bents are used to resist wind acting on the long sides of the building.

Vertical bracing is located in the exterior walls on column lines 1 and 4 to carry wind loads acting on the short side of the building. As an alternative, the exterior masonry walls can be used to resist wind on the short side of the building. The stiffness of these walls may resist a portion of the wind shear even if vertical bracing is provided.

The plastic design example considers the design of the supported Bents A and lateral force resistant Bents B shown on the floor plan in Fig. 9.1. The supported Bents A are designed to carry only gravity loads. Horizontal forces are transmitted from Bents A by diaphragm action of the floor slab to Bents B. The bending resistance of the members in each Bent B are assumed to resist the horizontal shears from wind on a 48 ft. length of the building and to provide the stiffness needed to resist the P-Δ effects.

5.4 SCOPE OF DESIGN EXAMPLE

The design example is organized into four parts:

Part 1: Design of supported Bent A for Gravity Load - Chapter 5

Part 2: Design of Lateral Force Resistant Bent B for Gravity and Combined Loads - Chapter 6

Part 3: Design Checks for Bents A and B—Chapter ~~6~~ 7

Part 4: Design of Typical Connections—Chapter ~~7~~ 8

The calculations are arranged in a tabular manual subroutine format, for ease of reference and to suggest the potential for computer subroutines. (Ref. ~~9.2~~ 13) A condensed form of the calculations can be adopted after attaining familiarity with plastic design. The manual subroutines used in each part of the design example are listed in Tab. ~~8.1~~ 9.1.

The emphasis in Parts 1 and 2 of the design example is on the selection of members to satisfy one or more design criteria which are likely to control. Design checks of the trial members for other pertinent design criteria are considered in Part 3.

The manual subroutines used in the design of Bent A include Tables ~~8.2~~ 9.2 to ~~8.8~~ 9.8 and are listed in Tab. ~~8.1~~ 9.1. The major steps in the design are summarized below.

1. Design the roof and floor girders for factored gravity load in Tabs. ~~8.2~~ 9.2 and ~~8.3~~ 9.3.
2. Tabulate column load data and gravity loads in the columns in Tabs. ~~8.4~~ 9.4 and ~~8.5~~ 9.5.
3. Determine the column moments for factored gravity load in Tab. ~~8.6~~ 9.6.
4. Select column sections for factored gravity load and investigate these sections for in-plane bending and lateral-torsional buckling under combined axial load and bending in Tabs. ~~8.7~~ 9.7 and ~~8.8~~ 9.8.

These steps are described in Arts. ~~4.5~~ 5.5 to ~~4.8~~ 5.8. The column design criterion is stated in Art. ~~4.7~~ 5.7 and reviewed in Art. ~~4.9~~ 5.9.

5.5 DESIGN OF GIRDERS IN BENT A (Tabs 9.2 and 9.3)

The roof girders for Bent A are selected in Tab. 9.2 and the floor girders in Tab. 9.3. The roof and floor girders in the two outside bays are fastened by a simple connection to the exterior columns and a rigid connection to the interior columns. The use of a simple connection will require a larger girder, but the fabrication and erection costs may be lower than for a girder with two rigid connections. For a comparison of the difference in girder sizes required, refer to Tabs. 8.2 and 8.3 of Ref. 1. The roof and floor girders in the interior bay are fastened by a rigid connection at both ends to the interior columns.

For the exterior roof and floor girders, the formation of 2 plastic hinges under uniformly distributed factored gravity loading (Tab. 9.2) results in the formation of a 3-hinged beam mechanism (Fig. 3.4). Using the clear span L_g, the required plastic moment is calculated as

$$M_p = \frac{FwL_g^2}{11.66} \qquad (5.1)$$

where w is the uniformly distributed working load on the girders, modified by live load reduction factors of the American Standard Building Code (Ref 9.3 Section 3.5) for the floor girders, and $F = 1.7$ is the gravity load factor.

For the interior roof and floor girders, the formation of 3 hinges under uniformly distributed factored gravity loading (Tab. 9.3) results in the formation of a 3-hinged beam mechanism. Again using the clear span L_g, the required plastic moment is calculated as

$$M_p = \frac{FwL_g^2}{16} \qquad (5.2)$$

The required plastic modulus $Z = M_p/F_y$ is used to select all girder sections.

If the exterior roof girders were rigidly connected at both ends, it would be assumed in Tab. 9.2 that the exterior columns below the roof would provide a plastic moment capacity (reduced for axial load) at least equal to that of the exterior roof girders. Article 6.3 of Ref. 9.4 describes a method for redesigning the exterior roof girders when the supporting columns have smaller plastic moment capacities than the girders.

The girders selected in Tabs. ~~8.2~~ and ~~8.3~~ are 9.2
adequate for factored gravity load. These trial 9.3
sections will be checked for live load deflection
and lateral bracing requirements in Chapter ~~6~~. 7

5.6 COLUMN GRAVITY LOADS AND MOMENTS-BENT A (Tab. 9.4)

The loading pattern that is likely to control the size of the columns in Bent A is full factored gravity load on all girders ($F = 1.7$). This article is concerned with the determination of the axial loads and moments in the columns for this loading condition. Other gravity loading conditions, consisting of various "checkerboard" live load patterns on alternate floors and bays, will produce different moment and end-restraint conditions in the columns. The effect of checkerboard loading on the columns is considered in Chapter ~~6~~. Here, it suffices to comment that checkerboard loading does not govern the column design in this example; it should be investigated when the adjacent girder spans and loads are nearly equal and the ratio of dead load to total load on these spans is less than 0.75.

The column design begins with Tab. 9.4 in which the column loads originating from the roof and from each floor are determined. The first 8 lines in this table are used to record tributary floor areas and unit loads. Line 1 of Tab. 9.4 assumes that the exterior and interior columns support 41.4% and 58.6%, respectively, of the area in the exterior bays. This distribution assumes the formation of a mechanism in the exterior girders (Fig., Tab. 9.2), which will occur under full factored gravity loads with F = 1.7. If no hinges formed in the exterior girders, which would be the case under working gravity loads, the exterior and interior columns might support about 37.5% and 62.5%, respectively, of the area in the exterior bays. Since the ratio .625/.586 = 1.07 is less than the gravity load factor F = 1.7, the distribution of 41.4% and 58.6% is the most critical. For preliminary design purposes a distribution of 40% and 60% could have been used. Had the exterior girders been rigidly connected at both ends, the usual distribution of 50% to each column would be valid.

Lines 9 to 13 include the calculations for the working load in the columns below the roof. Lines 19 to 22 give the total dead load and live load contributed by each floor.

The values are used to find the maximum percent live load reduction, Max. R in line 23 (Ref. 4, Section 3.5). The limiting value of the live load reduction is Max. R or 60 percent. Line

9.3 14

24 gives the percent live load reduction below level 2, based on the tributary floor area. When this rule is applied below level 4, it is found that the permitted live load reduction is controlled by the 60 percent limit from levels 4 to 24. The reduced live loads from the top three floors are entered in lines 27 to 29 of Tab. ~~8.4~~ 9.4. Line 30 gives the constant reduced live load increment from levels 5 to 24. The calculations in this table are independent of the design method since the same working loads are used in plastic design as in allowable stress design.

COLUMN LOADS (Tab. 9.5)

The column dead and reduced live loads are tabulated in Tab. ~~8.5~~ 9.5. The first line of numbers in this table is the load increment from one floor which is constant between levels 5 and 22. For example, the dead load increment of ~~34.6~~ 31.4 kips in Col. (1) is obtained from line 19 of Tab. ~~8.4~~ 9.4. The sum of the dead and reduced live loads gives the working loads in Cols. (3) and (8) of Tab. ~~8.5~~ 9.5. Multiplication by $F = 1.7$ and 1.3 yields the factored loads needed in the plastic design of the columns in supported Bent A.

COLUMN MOMENTS (Tab. 9.6)

The columns must also resist bending moments which are determined in Tab. ~~8.6~~ 9.6. The sign convention and notation for moments on a joint are indicated below the table. Positive moments act clockwise on the ends of members (or counter-clockwise on joints) and M_j denotes a moment about the center of the joint. The additional subscripts A and B indicate moments at the left and right ends of girders, while U and L denote moments at the upper and lower ends of columns. Equilibrium of moments on a joint is then expressed by the equation

$$\Sigma M_j = 0 \quad \text{or}$$

$$M_{jU} + M_{jL} = -(M_{jA} + M_{jB} + M_{je}) \quad (\cancel{4.2}\ 5.3)$$

where M_{je} is the moment about the center of the joint caused by eccentrically framed members such as the spandrel beams. The right side of this equation represents the net girder moment on the joint.

Full factored gravity load may be assumed to cause plastic hinges at the rigidly connected ends of all girders in Bent A. Thus the girders apply known moments to the joints. These girder moments do not depend on the joint rotations because the girder plastic hinges eliminate compatibility between the end rotations of the girders and columns. The sum of the column moments, $M_{jU} + M_{jL}$, above and below a joint is statically determined from Eq. ~~4.2~~ 5.3.

The moment at the center of a joint from the girder to the left of the joint is

$$M_{jB} = M_B + V_B \frac{d_c}{2} \tag{5.4}$$

where M_B is the girder end moment at the right end at the face of the column (M_B = Req'd M_p for rigid connections; $M_B = 0$ for simple connections), V_B is the girder end shear at the right end under full factored gravity loading (V_B up is positive), and d_c is the column depth. Similarly, the moment at the center of a joint from the girder to the right of the joint is

$$M_{jA} = M_A - V_A \frac{d_c}{2} \tag{5.5}$$

where M_A is the girder end moment at the left end at the face of the column (M_A = Req'd M_p for rigid connections; $M_A = 0$ for simple connections), V_A is the girder end shear at the left end at full factored gravity loading (V_A up is positive), and d_c is the column depth. V_A and $V_B = 0.5$ $(1.7\ wL_g)$ for a girder with both ends rigid, but they equal either

.414 (1.7 wL_g) or .586 (1.7 wL_g) (Table 9.2, figure at bottom) for a girder with one end hinged. Equations 5.4 and 5.5 are valid for any girder that forms a 3-hinged mechanism under uniformly distributed factored gravity loads.

Equations 5.4 and 5.5 are applied in lines 1 to 7 and lines 10 to 16 of Tab. 9.6.

The moments are then summed according to Eq. ~~4.2~~ 5.2 in lines ~~8~~ 9 and ~~16~~ 18. At the roof, $M_{jL} = 0$ so line ~~8~~ 9 gives the column moment M_{jU}. At joints below the roof, half of the net girder moment is distributed to the columns above and below the joint in line ~~17~~ 19. This distribution of column moments is a reasonable estimate but may be revised, if convenient, when the columns are designed. See Art. ~~4.9~~ 5.9. The results of the calculations in Tab. ~~8.6~~ 9.6 are summarized in the column moment diagram below the table, with moments plotted on the tension side.

~~4.7~~ 5.7 COLUMN DESIGN ASSUMPTIONS

The assumptions and design criterion for the columns in Bent A are discussed in this article. It is assumed that:

1. The ~~W~~ wide-flange columns are to be erected in two story lengths with their webs in the plane of the rigid bents.
2. Moments are applied only about the major axis of the columns, with no biaxial bending permitted. For this reason AISC Type 2 (simple) connections are used between the columns and the tie beams and spandrels.
3. Vertical bracing on column lines 1 and 4 at floor levels, or the stiffness of exterior walls, together with diaphragm action of the floor slabs, are considered adequate to prevent out-of-phase sidesway buckling of the rigid bents.

4. No lateral bracing is provided for the columns between floors. (This differs from the assumption of laterally braced columns in Ref. 4). 9.4 7
5. Moment resistance at the column bases is conservatively neglected in the design of the bottom story columns.

6. Column sizes may vary from 8 to 14 inch wide-flange shapes. This will require more expensive column splices between columns of two different nominal depths, but this extra expense may be offset by the extra saving in steel that will result. In the exterior columns especially the splices may not be much more costly, as these columns carry very little moment, and the splices can be designed on this basis.

The columns resist concurrent axial load and bending moments and are designed as beam-columns. Chapter 3 lists the parameters that may influence beam-column behavior, which include:

1) The end-moment ratio q, described in Fig. 3.12. This is an important parameter in the design of beam-columns, because of its influence on the end moment versus end rotation behavior (M-Θ). Table 9.6 indicates that under full factored gravity load all the columns in Bent A are bent in double curvature (q = +1.0), except those in the top and bottom stories where the end-moment ratio q = 0 is conservatively assumed.

2) Major axis slenderness, h/r_x. Large values of h/r_x may result in in-plane bending of the beam-column.

3) Minor axis slenderness, h/r_y. The h/r_y ratio is used to check for lateral torsional buckling of the beam-column.

The sum of the beam-column moment capacities above and below a joint must equal or exceed the net girder moment on the joint from Eqs. ~~4.2~~ and ~~4.4~~. This is the criterion to be satisfied in the design for full factored gravity load. The range of application of this column design criterion depends on the M-θ behavior of the beam-columns. This criterion will be ~~discussed~~ after the columns have been ~~designed~~.

5.4
5.5
investigated
selected

It is not necessary to apply the column design criterion for full factored gravity load at every joint in Bent A because of the equal floor loads and because the columns are erected in two story lengths. When the upper and lower segments of one column length have the same unbraced height and end moment ratio, the lower segment will provide the smaller beam-column moment capacity because this segment resists the larger axial load. This lower column segment can be designed to resist half of the net girder moment on the floor above the column splice. The top columns should be checked below the joints on level 2 and at the roof since the segments below the roof are not bent in double curvature.

~~4.8~~ DESIGN OF COLUMNS IN BENT A

5.8
(Tabs. 9.7 and 9.8)

Trial A36 column sections can be selected using the formula

$$P_y = P + 2.1\,M/d \text{ but not less than } JP \qquad (\cancel{4.5})$$

5.6

where P = required axial load capacity, kips
M = required major axis end moment capacity, kip-ft.
d = estimated column depth, ft.

$P_y = AF_y$, kips

$J = 1.12$ for $F_y = 36$ ksi and $h/r_y \leqslant 40$

$= 1.18$ for $F_y = 50$ ksi and $h/r_y \leqslant 40$

J = 1.33

This formula assumes that the beam-column moment capacity is governed by M_{pc} from Eq. ~~3.3~~ 3.9a and is derived as follows:

Using $M_{pc} = M$ in Eq. ~~3.3~~ 3.9a gives

$$P_y = P + M\,(0.85\,P_y/M_p) \qquad (\cancel{4.6})\ 5.7$$

The ratio M_p/P_y may be expressed as a function of the depth d in the form

$$\frac{M_p}{P_y} = \frac{ZF_y}{AF_y} = \frac{Z\ 2d}{S}\left(\frac{r_x}{d}\right)^2 \qquad (\cancel{4.7})\ 5.8$$

Then Eq. ~~4.5~~ 5.6 follows from the approximations for most *WF* shapes, bent about the major axis

$$Z/S \approx 1.12 \qquad (\cancel{4.8})\ 5.9$$
$$r_x/d \approx 0.43$$

The term $2.1M/d$ in Eq. ~~4.5~~ 5.6 represents an "axial load equivalent" for the major axis moment. When this term is small compared with P the resulting P/P_y ratio approaches unity and the beam-column moment capacity is controlled by *lateral-torsional buckling*, instead of M_{pc}. See Design Aid III.

The basis for the qualification of $P_y = 1.33P$ is a result of the requirement that $P/P_y \leq 0.75$ for beam-columns (Art. 3.5a).

Trial columns sections for Bent A are selected and checked in Tab. 9.7 for the exterior columns and Tab. 9.8 for the interior columns. To facilitate the complete design and checking procedure without requiring the use of two tables for each column, note that two lines are used for each column in Tabs. 9.7 and 9.8. The procedure is as follows:

1) Enter the required axial load, P, and required axial moment, M, in Col. (1).

2) Enter the estimated column depth, d, and calculate the equivalent axial load, 2.1 M/d, in Col. (2).

3) In Col. (3) calculate the required P_y value based on Eq. 5.5, and select a trial section based on this value.

4) Enter the column height, h, and calculate the column end-moment ratio, q, in Col. (4).

5) In Cols. (5) and (6) enter the P_y, M_p, r_x, and r_y values for the trial section, and in Col. (7) calculate the major and minor axis slenderness ratios, h/r_x and h/r_y, respectively.

6) Calculate the P/P_y value for the trial section in Col. (8), and from Eq. 5.6 calculate the maximum allowable M_{pc}/M_p ratio.

7) In Col. (9) use Design Aid III to find the beam-column moment capacity in the form M/M_{pc}, as limited by lateral-torsional buckling and in-plane bending.

For values of q between +1.0 and 0, or between 0 and −1.0, conservative estimates of M/M_{pc} can be obtained from the charts for q = 0 or −1.0 respectively.

The maximum allowable moment capacity $M = (M/M_{pc}) \times (M_{pc}/M_p) \times M_p$ is finally calculated and recorded in Col. (9). The trial section is adequate for full factored gravity load if the allowable moment capacity is at least equal to the required moment. The decision as to adequacy is noted in Col. (10).

It should be noted that the large minor axis slenderness ratios h/r_y in the exterior columns below levels R and Z result in lateral-torsional buckling failure of the trial columns selected. The sections selected will, however, reach the required moment capacity before lateral-torsional buckling occurs. Notice that only in Tab. 9.8, below level R, is it feasible to try a lighter column section to replace the initial column section. For sections governed by the condition $P_y = 1.33 \times P$, a lighter section can obviously not be used.

In the lower protion of Tabs. 9.7 and 9.8, alternate designs are included using A572 steel in the lower columns. The slenderness ratios for the A572 steel columns are modified by the coefficient $F_y/36$ as per Eq. 3.11. The use of A572 steel in the interior columns may be especially economical, because all the interior columns will be of the same nominal depth (12 in.).

An assumption used in the column design for Bent A is that 50 percent of the net girder moment on a joint is distributed to the column below. This assumption is conservative if the columns above and below the joint have similar P_y, slenderness, and end moment ratios. Under these conditions, the peak moment of the column above, M_{jL}, is less than the peak moment of the column below, Mju, because the lower column resists a larger axial load (CASE 1 - Fig 5.1).

In the linear elastic portion of the M-θ curves, the M-θ slope determines the column moment distribution. The elastic M-θ slope depends on the column stiffness I/h and the end moment ratio q. The effect of a plastic plateau in the column M-θ curves is to redistribute the column moments in proportion to M_{pc} as shown in Fig. 4.1(c). The column design can be modified to take advantage of this plastic behavior by assuming that the column below a joint resists less than 50 percent of the net girder moment. If the *sum* of the beam-column moment capacities above and below the joint is at least equal to the net girder moment, the design is adequate.

The procedure for determining the maximum column moment sum when LTB limits the joint rotation and moment capacity of one column
(Case ~~5~~, Fig. ~~4.2~~), can also be applied when the 2
LTB moment for one column is less than 50 5.1
percent of the net girder moment on the joint. Redistribution of column moments is a design refinement in the direction of economy but it is not a mandatory design requirement.

The results of the tentative design of supported Bents A are summarized in Fig. ~~8.2~~. 9.2
These are checked in Chapter ~~6~~. 7

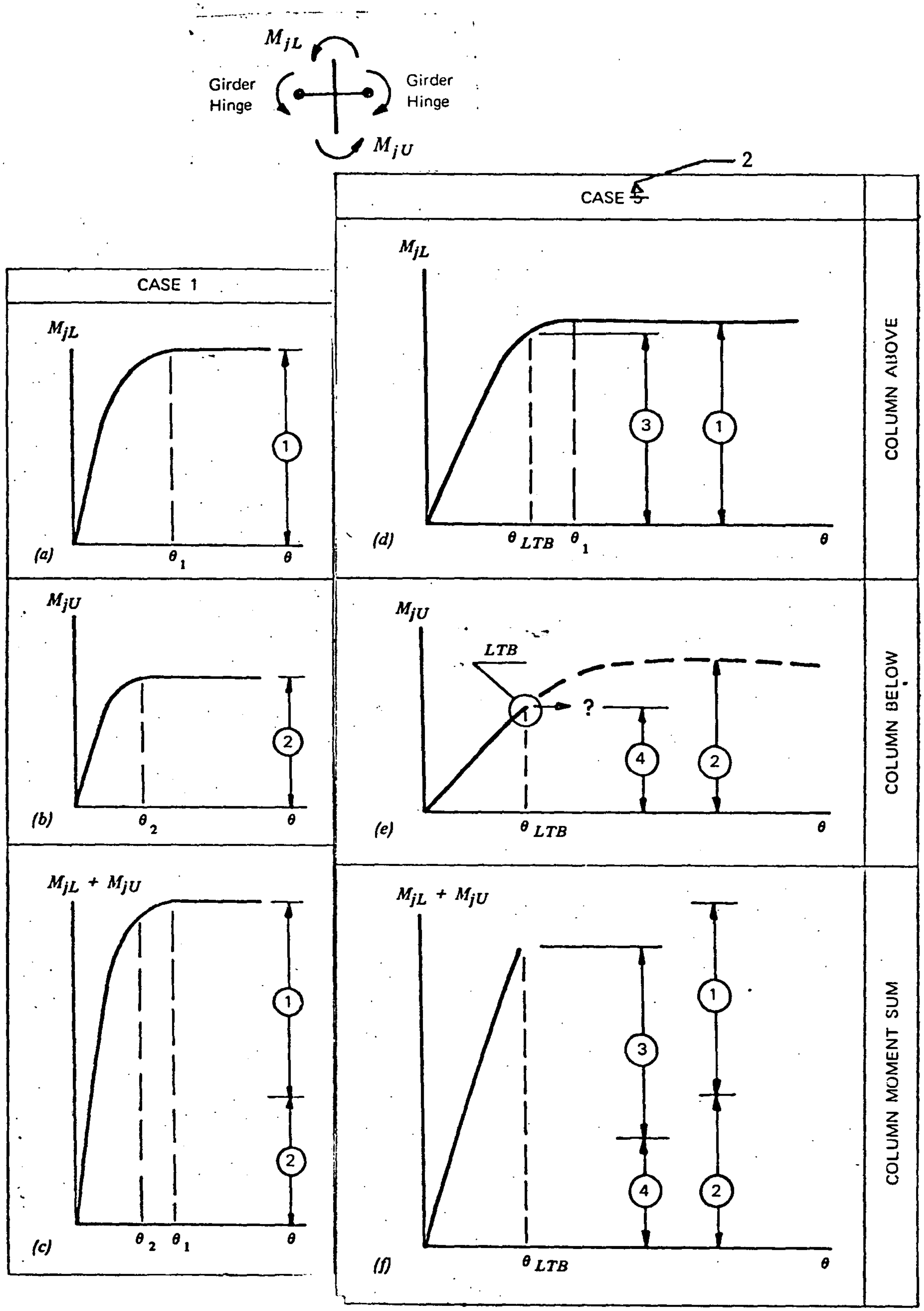


FIG. ~~4~~.1 INFLUENCE OF M-θ CURVES ON MAXIMUM COLUMN MOMENT SUM 5

CHAPTER 6

DESIGN OF BENTS FOR GRAVITY AND LATERAL FORCE

6.1 Introduction

This chapter illustrates the design of unbraced Bent B in the multistory building described in Chapter 5 and Fig. 9.1. This is part 2 of the design example.

Bent B must provide adequate strength to resist both factored gravity loading and factored gravity plus lateral loading. It must also be stiff enough to keep the horizontal drift within acceptable limits.

Selection of members may be governed by either strength or stiffness. The designer must specify criteria for both and base the design on these requirements as well as check the design for remaining requirements.

The design of Bent A, described in Chapter 5, illustrates many features of the plastic design method applicable when strength is the controlling criterion. Unlike a braced frame, an unbraced frame develops its strength and stiffness from the beams and columns alone. The design of unbraced Bent B in this chapter indicates how the stiffness criteria cause an increase in the needed member sizes. Chapter 7 describes methods for checking design requirements not used in selecting member sizes.

The slenderness of the framing system for Bent B is an important parameter which indicates whether strength or stiffness requirements will govern the design. The term slenderness refers to the ratio of the total height of the bent h_t to the distance L between exterior columns. For Bent B, h_t = 236.7' and L = 66', so h_t/L = 3.6. This ratio suggests that the frame is relatively slender and that stiffness may be an important factor, especially in the middle and lower regions.

The manual calculation procedures used in the design of Bent B include Tabs. 9.9 to 9.27 and are listed in Tab. 9.1. The major steps of the design are listed below.

1. Tabulate column gravity load data in Tabs. 9.9 to 9.11.

2. Design the floor girders for factored gravity plus lateral load. This step is referred to as the combined load statics calculation and is performed in Tabs. 9.12 and 9.13.

3. Bring moments at joints into equilibrium by moment balancing in Tabs. 9.14 to 9.18.

4. Determine column thrusts for combined loading, Tabs. 9.19 to 9.20.

5. Determine column gravity load moments, Tab. 9.21.

6. Select columns in Tabs. 9.22 to 9.27 in accordance with the requirements of Chapter 3.

7. Check story rotation, drift, and secondary design criteria in Chapter 7.

6.2 Column Gravity Loads (Tabs. 9.9, to 9.11)

This article is concerned with the determination of the axial loads in the columns under full factored gravity loading in all columns, with F equal to both 1.3 and 1.7. As discussed in Art. 5.6, other gravity loading conditions will produce different column loads. The effects of these load patterns are considered in Chapter 7, and will be considered here.

Unlike the exterior girders in Bent A, all girders in Bent B are fastened by a rigid connection at both ends to their respective columns. This condition is not necessary. It has been found, however, that the use of even only one or two simple connections per story makes it impossible to control deflections in a lateral load carrying bent unless exceptionally large girders are used.

The calculation of the column axial loads begins in Tab. 9.9, in which the working loads for the roof and floor girders are listed. Tab. 9.9 also lists joint wind load data for future reference. In Tab. 9.10 the column loads originating from the roof and from each floor are determined. The calculations in Tab. 9.10 correspond to those made in Tab. 9.4 for Bent A. However, in Lines 1 and 2 a distribution of 50% is used for both the exterior and interior girders as they are rigidly fastened at both ends. The common increments in lines 19 and 30 are entered in boxes at the top of Tab. 9.11.

The column dead and reduced live loads are tabulated in Tab. 9.11. Multiplication by F = 1.7 and 1.3 yields the factored loads needed in the plastic design of the columns.

6.3 Combined Load Statics Calculation (Tab. 9.12)

In Tab. 9.12 equilibrium of the frame in its displaced position is formulated as was discussed in Chapter 4.

Col. (1) tabulates the wind load acting on Bent B. The wind forces are calculated assuming that a lateral pressure of 20 psf., acting on a 48 ft. width of the building, is concentrated at each floor level. In Col. (2) the cumulative wind load is multiplied by the combined load factor (F = 1.3) to find the factored wind shears immediately below each level.

The $P\Delta/h$ shears are proportional to the total factored gravity load ΣP, for both Bent B and the supported Bent A. It is assumed that the $P\Delta/h$ shears created in the supported Bent A are transferred by diaphragm action of the floor slabs to Bent B. It is obvious that if Bent A has any stiffness against lateral load, which it undoubtedly has, then it will help to resist a portion of its own $P\Delta/h$ shear. This assumption of complete $P\Delta/h$ shear transfer to Bent B is conservative. In calculating the $P\Delta/h$ shears, an ultimate load drift index, (Δ/h)ult., must be assumed. As noted in Tab. 9.12, (Δ/h)ult. is assumed as 0.010 for stories 2 to 24. To give a somewhat conservative design, these values are obviously larger than the expected ultimate drift.

The total factored shears immediately below each level are given in Col. (7). This horizontal shear below each floor level is needed to calculate the sum of story column end moments and the sum of girder end moments required to resist the combined gravity plus lateral load applied to the frame. Later, these end moment sums are distributed to find the end moments on each member. The sum of column joint moments for one story of Fig. 4.1 is calculated in Col. (8) of Tab. 9.12; it equals the total factored shear of a floor level multiplied by the story height as expressed in Eq. (6.1)

$$\Sigma M_c = \Sigma H + \Sigma P \left(\frac{\Delta}{h}\right)_{ult} \times h \qquad (6.1)$$

Therefore, it is obtained by multiplying each shear value in Col. (7) by the appropriate story height. This sum of story column joint moments is listed for the set of columns below each indicated floor level.

As an estimate for preliminary selection of members, it is assumed that half of the total column moments are at the top and bottom of each set of columns in a story. Then, the sum of clockwise end moments on all beams of a level is approximately

$$\Sigma M_g = -\tfrac{1}{2} \quad (\Sigma M_c)_{lev.\ n-1} + (\Sigma M_c)_{lev.n} \qquad (6.2)$$

in which lev. n-1 refers to the column end moments in the story above the floor level and n refers to the column end moments below. The sum of girder end moments is calculated in Col. (9) in which each entry is the average of the values in Col. (8) for the given level and for the level above it.

Using the above mentioned estimate is the same as assuming an inflection point at the midheight of each column. From this assumption, the sum of story column end moments is distributed one-half to the top of the set of columns and the other half is distributed to the bottom of the set of columns.

This calculation is given in Cols. (8), (10), and (11). Each line of Col. (8) lists the sum of column moments in the story which supports the level. The assumed distribution factor (in this case ½) divides this sum into two portions related to the tops and bottoms of the columns. The portion related to the tops of columns is listed in Col. (11) on the same line as the given level and is identified as the moments below the level. The portion related to the bottoms of columns is listed in Col. (10) in the next lower line, where it is identified as moments above the lower level. This sequence is followed down through the table.

The bottom story could require special consideration, depending on the column base detail. In this example the foundation is designed to provide fixed bases. Therefore, the assumed distribution factor (again ½) divides the sum of column end moments equally to the top and bottom of the bottom columns as indicated in Col. (11).

6.4 Design of Girders (Tab. 9.13)

The design of floor girders for Bent B is given in Tab. 9.13. The requirement for the design of girders is a knowledge of the end moments on the girders. The sum of girder joint moments for a floor level was calculated in Tab. 9.12. The sum of girder joint moments for each bay is calculated in Tab. 9.13 by distributing to each girder a percentage of the total sum of girder end moments for a floor level. Although this distribution is arbitrary, for preliminary design it is believed that an equal division of the sum of end moments will require more nearly equal sizes for all girders on a level. Therefore, the distribution factor used here is 1/3 for the three bays of Bent B, which results in an equal sum of girder end moments M_g for each girder of one level.

The steps to obtain a preliminary girder section are summarized below and are found from the statical relationships given in Chap. 4.

1. Calculate M_{pm} required for each girder. Because of the difference between roof and floor loads M_{pm} for lev. 1 for each bay differs slightly from the equal values of M_{pm} for levs. 2 - 24.

For each girder calculate the required clear span sway moment coefficient G.

$$G = M_g \times \frac{(1-\frac{dc}{L})}{M_{pm}} \qquad (6.3)$$

2. Calculate the plastic hinge moment ratio R using either

$$R = (\frac{2}{C+1}) \quad x \quad (1 + \frac{G}{8})^2 \quad \text{for } G \leq 8 \qquad (6.4)$$

or $$R = (\frac{1}{C+1}) \quad x \quad (G) \qquad \text{for } G \geq 8 \qquad (6.5)$$

where C is the positive plastic moment coefficient used to restrict hinge formation and also to limit drift under combined loading. C will be specified to be 0.4 for the combined loading case, thereby limiting the maximum positive moment in the girders to 0.4 M_p. C has been specified to such a small value for the combined loading case in an effort to control sway deflections in the relatively slender Bent B.

3. Calculate R_{LB} which is the value of R used for gravity loading.

$$R_{LB} = (\frac{2}{C+1}) \quad x \quad (\frac{F_1}{F_2}) \qquad (6.6)$$

For the gravity load case, C is premitted to reach its maximum value of 1.0, since sway deflections need not be controlled under gravity loading.

4. Find the required plastic moment M_p for each girder.

$$M_p = R \times M_{pm} \text{ when } R > R_{LB} \qquad (6.7)$$

$$M_p = R_{LB} \times M_{pm} \text{ when } R < R_{LB} \qquad (6.8)$$

It is interesting to note that for the particular case of $C = 1.0$ for gravity loading, R_{LB} will be less than R for all values of G. This indicates that for Bent B all girders will be controlled by the combined loading case. Under circumstances where drift is not a problem and $C = 1.0$ is used for both loading cases, then the girders in the upper levels will be controlled by gravity loading.

5. A required plastic modulus Z for each girder is calculated from the equation

(6.9)

with appropriate units for M_p and F_y. Cols. (5) and (9) of Tab. 9.13 list the required Z values for all the girders. Preliminary girder sizes can then be selected from DA-I using these Z values. Preliminary girder sizes are indicated in Fig. 9.3.

6.5 Girder Clear Span End Moments And Joint Moments (Tab. 9.14)

From the required moment capacity of the girders, the values of girder clear span end moments and girder joint moments are calculated for the combined loading case in Tab. 9.14. The clear span end moments are used in the design of the girders. These end moments tranformed to the column center lines, called joint moments, are used in the design of the columns.

INSERT FIG. 6.1

The criterion used for designing the girders under combined loading is shown in Fig. 6.1. For C = 1.0, the girders form a 2-hinged mechanism. For C < 1.0, a hinge forms only at the leeward end, called end B, the maximum positive moment being equal to C X M_p. From Fig. 6.1 it is seen that the leeward clear span moment on the girder, M_B, is always equal to the plastic moment M_p for the section. The windward clear span moment on the girder M_A equals the total sway moment on the girder minus the leeward moment value.

$$M_A = G \times M_{pm} - M_B \qquad (6.10)$$

The moments at the center of the joints are calculated in Tab. 9.14 using the sign convention of Fig. 4.2. The joint moments are found using the equations

$$M_{jB} = M_B + (4 + \tfrac{1}{2}G) \times M_{pm} \times \left(\frac{\frac{dc}{L}}{1-\frac{dc}{L}}\right) \qquad (6.11)$$

$$M_{jA} = M_g - M_{jB} \qquad (6.12)$$

6.6 Moment Balancing (Tabs. 9.15 to 9.18)

After defining girder joing moments which help resist the horizontal forces applied to the frame, the determination of column joint moments which are in equilibrium with the girder joint moments is executed in Tabs. 9.15 to 9.18. One table is presented for each of the four columns. Due to the symmetry of Bent B, the moment balance procedure is executed for only one direction of lateral force. This still requires knowledge of all the column end moments, therefore, the method is worked for all columns of each floor level. If the geometry or the loading were unsymmetrical, the moment balance procedure would have to be carried out for both directions of lateral force.

The initial joint moments for the columns are calculated in Cols. (1) and (2). They are initialized by distributing the sum of column joint moments of Tab. 9.12, Cols. (10) and (11), among the columns. For Bent B the distribution factor is assumed to equal 0.25 for each of the four columns. Therefore, the sum of the column end moments is divided and assigned equally to each column. As indicated in Cols. (1) and (2) of Tabs. 9.15 to 9.18, for each particular joint there are column joint moments marked "above" and "below". The column moment marked "above" is the joint moment of the column above the floor level at the lower end of the column. The column joint moment marked "below" is the joint moment of the column below the floor level at the upper end of the column.

The unbalance in joint moments at each joint for a floor level is obtained by summing the right end moment from the girder left of the joint, the left end moment from the girder right of the joint, and the column joint moments above and below the joint. The reverse sign of this unbalanced moment is the amount which must be added to balance the joint. It is observed that the sum of the unbalances of all joints in a level is zero. This means that the proper amount of column moment is present, but some of it is temporarily assigned to the wrong columns.

The joints are balanced by distributing the unbalance to the column ends above and below the joint. For Bent B a distribution factor of 0.5 is used for distributing the unbalance above and below the joint. For most frames this distribution of 0.5 will be sufficiently accurate, even though the column below the joint may take a slightly larger portion of the unbalance, since the lower column may have a higher stiffness than the upper column. The changes in column joint moments are then added to the initial column joint moments to give the final column joint moments.

As a result of this moment balancing procedure a set of column joint moments are now determined; they are in equilibrium with the girder joint moments and with the lateral shears due to wind and P-delta effects. They may be used as a basis for preliminary selection of column members.

6.7 Procedure to Find Foundation Moments (Tabs. 9.15 to 9.18)

The moment balance for the bottom story may follow the same procedure as that used in the stories above. Knowledge of the moment resistance of the column base detail is necessary to do the moment balance. The foundations in this example are designed to provide full base fixity. Therefore, the increment of moment at the top of the bottom story column, due to moment balancing, results in an additional increment of moment at the column base. For this example where full base fixity is assumed, one-half of each additional bottom story column top moment is carried over to each column base.

The moment balance procedure for bottom story column moments is done in the lower part of Tabs. 9.15 to 9.18. The sum of initial column base moments is given in Tab. 9.12. Then, the sum of column base moments is divided and assigned equally to each bottom story column, again using a distribution factor of 0.25 for each of the 4 columns.

The unbalanced moment applied to the top of the bottom story column is then carried-over to the column base, resulting in the additional foundation moment. The foundation moment is then the sum of the initial and carry-over moments.

6.8 Column Thrusts For Combined Loading (Tabs. 9.19 and 9.20)

The column thrusts for combined gravity plus lateral load are calculated for each line of columns in Tabs. 9.19 and 9.20, one table for exterior and one for interior columns. Due to the geometric symmetry of of Bent B, the column thrusts are found for only one direction of lateral force. This still requires knowledge of all column thrusts. If the geometry or the loading were unsymmetrical, the column thrusts would have to be found for both directions of lateral force.

The determination of the thrusts is based on calculating the vertical shear in the girders adjacent to a column and algebraically adding this shear to the thrust based on gravity loading of the tributary area. Due to symmetry of Bent B, the exterior column axial forces are equal for gravity loading, and the interior column axial forces are equal for gravity loading. The vertical shear for a girder to the left of a column is added to the gravity axial load because the net girder wind shear results in a downward force. The vertical shear for a girder to the right of a column is subtracted from the gravity axial load because the net girder wind shear results in an uplift.

Vertical shear is determined by dividing the sum of end moments for each girder by the girder span length. In Art. 6.4, the total sum of girder end moments for each floor level was evenly distributed to each girder. Therefore, every girder per floor level has an equal sum of end moments. In addition, the vertical shears of girders for bays 1 and 3 are equal due to symmetry. These were previously listed in Col. (1) of Tab. 9.13, and for convenience they are relisted in Col. (1) of Tabs. 9.19 and 9.20.

The $\Sigma(2)$ in Col. (3) of Tabs. 9.19 and 9.20 indicates the sum of the girder shears from Col. (2). In Tab. 9.19, the cummulative girder shears of bays 1,3 are subtracted and added to the factored (F = 1.3) column gravity thrusts for the left exterior and right exterior columns respectively. In Tab. 9.20, the cumulative girder shears at bays 1,3 are added and subtracted to the left interior and right interior columns, respectively, while the cumulative girder shears of bay 2 are subtracted and added to the left interior and right interior columns, respectively.

The column thrusts have been computed for the combined loading case (F = 1.3). They are used in determining the required column sections.

6.9 Column Gravity Load Moments (Tab. 9.21)

Column end moments due to factored gravity loading (F = 1.7) are calculated in Tab. 9.21. These moments are needed to make a comparison with the moments for combined loading to determine which loading condition controls the column design.

As indicated in Art. 6.4, the girder member sizes for Bent B are all controlled by the combined loading condition. Because these girder sizes are larger than those which would have been required to carry only gravity loads, it would be unrealistic to assume gravity load end moments of the girders equal to the plastic moment of the section. (As previously mentioned, girders in the upper 2 to 3 stories may be controlled by gravity loading, for which the plastic moment of the section would occur at the ends.). Therefore, the gravity load moments are assumed to equal the elastic fixed end moments on the girder clear span and are given as

$$M_B = -M_A = \frac{1}{12} F_1 w L_g^2 \qquad (6.13)$$

The end moments are given in Cols. (1) and (5) for bays 1,3 and bay 2, respectively.

The joint moments due to these end moments are then calculated as

$$M_{jB} = -M_{jA} = M_B + 4 \times \frac{F_1 \, wL_g^2}{16} \times \frac{dc/L}{(1-dc/L)} \qquad (6.14)$$

The sum of the girder end moments, considering the girders to the left and right of the joint, are calculated in Cols. (2) and (6) for joints 1 and 2, respectively. Due to symmetry of Bent B the end moments for joints 3 and 4 are not shown. The sign convention for girder joint moments is indicated in Fig. 4.2.

From equilibrium, the sum of girder joint moments equals the sum of column joint moments at each joint. To find each column joint moment for the gravity loading case, the girder joint moment sum is assumed to be distributed equally to the column above and below the joint. Column joint moments above and below the joint are given in Cols. (3), (4), (7) and (8). These column joint moments will be used in a later table to calculate the column end moments under gravity loading.

6.10 Design of Columns in Bent B (Tabs. 9.22 to 9.27)

The design of columns in Bent B follows the same procedure as the design of columns in Bent A, Art. 5.8. However, the columns of Bent B must be designed for both the combined and gravity loading conditions, whichever is the most critical.

Tabs. 9.22 to 9.24 are used to select a preliminary column size. The first step is to determine the column end moments from the previously determined column joint moments. For any given column joint moment M_j, the column end moment M_c may be calculated as

$$M_c = M_j - V \frac{d_g}{2} \qquad (6.15)$$

where d_g is the depth of the girder, and V is the column horizontal shear. For a given column, V is calculated as

$$V = \frac{M_{ju} + M_{jL}}{h} \qquad (6.16)$$

where M_{ju} and M_{jL} are the upper and lower column end moments, respectively, and h is the center-to-center height of the column. Using Eqs. 6.15 and 6.16, the column end moments for a column below a level are given in Cols. (6) and (7) of Tabs. 9.22 to 9.24.

Trial column sections for major axis bending can then be selected using Eq. 5.5

$$P_y = P + 2.1\ M/d \qquad (5.5)$$

where P is the required axial load capacity, and is listed in Col. (8) for the appropriate loading condition of each of Tabs. 9.22, 9.23, or 9.24. As before, P/Py must be less than 0.75. The column depth d has been assumed equal to 14 inches for Bent B. The major axis bending moment is the largest moment in the column. Since the columns in Bent B are bent in double curvature, the largest moment in a column is either at the top or bottom end of the column.

As discussed in Art. 5.7 in relation to Bent A, it is not necessary to apply the column design criteria at every level in Bent B. The same column levels designed for Bent A are designed for Bent B, as indicated in Tabs. 9.22 to 9.24. In these tables notice, however, that to obtain the bottom column end moment for the column below n, it is necessary to know the column end moment above level n + 1.

Using DA-I, trial sections are selected to satisfy the trial P_y values selected in Col. (9). Trial sections are selected only for the critical loading condition. It is of interest to note that gravity loading controls the design of the columns only in the upper stories.

As for Bent A, the lower story columns of Bent B are also designed using A572 steel.

Preliminary column sizes are then checked in Tabs. 9.25 to 9.27 for their in-plane bending strength and lateral-torsional buckling strength using DA-III. If the allowable moment is greater than the required moment, and the ratio P/P_y is less than 0.75, then the column section is sufficient. Lighter trial sections can be tried, to guarantee that the chosen column section is the most economical. Refer to Art. 5.8 for a more detailed explanation of these tables.

The columns in Bent B are limited to 12" and 14" sections. Smaller depths in the upper story columns may result in lighter sections, but this would entail additional splices between columns of different depths. An additional consideration which must be taken into account is that below level 2, a lighter column could be used than that required below level 1. This is because of the large column moment below the roof level. However, it is better practice to forego a nominal weight saving by running the same size column straight through, rather than making a splice from a heavier column above to a lighter column below.

Preliminary column sizes for both A36 and A572 steel are indicated in Fig. 9.3.

CHAPTER ~~6~~ 7

Design Checks and Secondary Considerations

~~6.1~~ 7.1 INTRODUCTION

The primary stage of a structural design is usually concerned with the proportioning of members for strength and/or stiffness. The design conditions considered are full factored gravity loading, factored combined loading, and wind drift. In Chapter ~~5 drift~~ 6 an assumed value for the sidesway of the frame was chosen ~~as the governing design condition for the columns in Bent *B*. In this chapter the strength of these columns will be checked for gravity load ($F = 1.7$) and combined load ($F = 1.3$).~~ to give a somewhat conservative design. First of all a check must be made to assure that the chosen sway is definitely adequate to check for the stability of the frame, and to assure that the sway at working load remains within an acceptable limit.

In addition there are secondary conditions (secondary meaning that they are not usually considered in the initial design) that may govern the design of individual elements in the structure. These conditions are:

1. partial live or "checkerboard" loading
2. deflections at working load
~~3. sidesway under factored gravity load~~
3. ~~4.~~ spacing of lateral bracing
4. ~~5.~~ effect of shear on bending capacity
5. ~~6.~~ uplift at footings

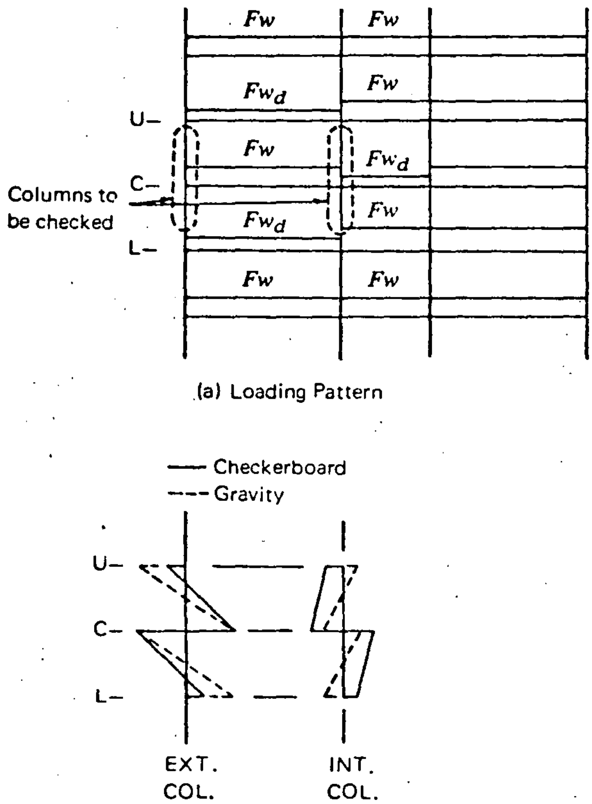
The approach used for these design checks is to make conservative assumptions and approximations in order to find out if there is a problem in the first place. If the preliminary conservative calculations do not satisfy the particular requirement, then more careful analysis is performed. The idea is that usually the secondary design situations are not critical, so they do not warrant undue design time.

7.2 SWAY ANALYSIS

No detailed calculations will be made here regarding the calculation of sway deflections. A more detailed discussion of sway analysis is given in Art. 4.4.

~~6.3~~ CHECKERBOARD LOADING 7.3

The columns of Bents A and B can safely carry the full factored dead and live loads on all the stories. Full loading usually causes the columns to bend in double curvature ($q = + 1.0$),



(b) Moment Diagrams

FIG. ~~6.1~~ CHECKERBOARD LOADING 7.1

and the strength of a column is highest when bent in this configuration. If the ratio of the end moments q is reduced ($q < + 1.0$), the column strength can be adversely affected. See Design Aid III.

A situation more critical than full loading can develop if the factored live load is removed at a few locations. The typical loading arrangement that should be considered is shown in Fig. ~~6.1~~(a). The factored live loads are removed in alternate bays at levels U,C and L only. The column moments caused by this localized checkerboard arrangement tend to approach the most critical single curvature case ($q = -1$) while keeping the axial load relatively unchanged. The possibility of having a complete checkerboard pattern is extremely remote and not even as critical a condition, since axial load in the column would be substantially reduced. In the 7.1

"localized" checkerboard loading, the axial load will be reduced slightly, but at the lower stories the reduction is usually insignificant. A comparison between the moment diagrams for full gravity and checkerboard loadings is shown in ~~Fig. 6.1~~(b). Not only can q be reduced from the double curvature case, but the moment applied to the columns at Level C can be increased. 7.1

To evaluate the strength of a column under checkerboard loading, the end moments and q must be determined. In the full loading case, Eq. ~~4.3~~ 5.3 was used to calculate the net girder moment where M_B was taken as the required M_p ~~from Eq. 4.1~~ assuming rigid connections in case the full gravity loading governs the design. The net girder moment at the column centerline for checkerboard loading can be determined from Eq. ~~6.1~~ 7.1.

Net girder moment =

$$\pm\left[M_p + \frac{FwL_g d_c}{4}\right]_{\text{FULL LOAD}} \mp \left[M_d + \frac{Fw_d L_g d_c}{4}\right]_{\text{DEAD LOAD}} \qquad \text{~~(6.1)~~ } 7.1$$

where M_d is the moment at the ends of the girder under factored dead load alone and is assumed as

$$M_d = \frac{Fw_d L_g^{\,2}}{12} \qquad \text{~~(6.2)~~ } (7.2a)$$

for girders, the ends of which are rigid-connected and

$$M_d = \frac{Fw_d L_g^{\,2}}{8} \qquad (7.2b)$$

for girders with far end simply supported.

but may not exceed M_p. The sign convention is shown in Tab. ~~8.21~~ 9.27. If $M_d = M_p$, plastic hinges form at the ends of the girders under factored dead load alone, so there is no significant difference between checkerboard loading and full loading. In the design check, it is conservative to assume that only the columns resist the net girder moment and that this moment is equally divided between the columns framing into the joint. Once the column end moments are evaluated, q is calculated, and the column strength determined.

In the *exterior* columns, checkerboard loading only affects q since the column moments at the floors above and below the Level C under consideration are reduced while the moment at Level C remains constant as shown in Fig. ~~6.1~~(b). 7.1

However, q must be greater than zero in many cases because of the restraining effect of the members at the levels above and below. Therefore, it is conservative to use $q = 0$ for the exterior columns. A comparison of Design Aids III–1b and III–2b shows that for $h/r_y < 25$ there is no difference between the major axis bending strengths for $q = +1.0$ (double curvature) and $q = 0$ (one end pinned) unless $P/P_y > 0.6$. In fact, the difference does not become significant ($\sim$ 5%) until $P/P_y > 0.9$.

A reduction in q from +1.0 to 0 can also affect the lateral-torsional buckling strength (LTB). A comparison of Design Aids III–1a and III–2a shows when this change in q has an effect, and the results are given in Fig. ~~6.2~~ 7.2. Combinations of P/P_y and h/r_y that fall below the curve indicate that when $+1.0 < q < 0$, there will be no change in LTB strength (actually there will be no lateral torsional buckling). If values fall above the line, further analysis is indicated; Design Aid III–2a must be used to check for the actual LTB strength.

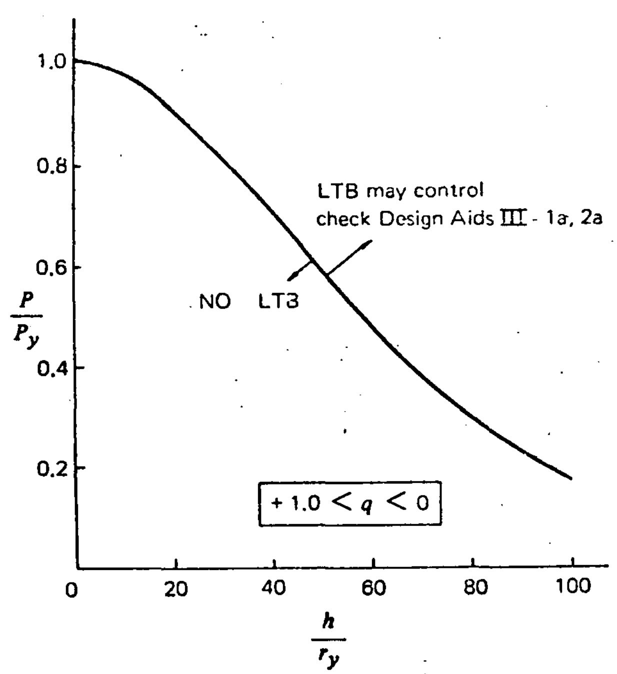


FIG. ~~6.2~~ 7.2 EFFECT OF LATERAL-TORSIONAL BUCKLING ON BEAM-COLUMN STRENGTH

In the *interior* columns, checkerboard loading affects both q and the maximum bending moment, and q can vary over the full range of +1.0 to −1.0. The curves in Design Aid II indicate that most columns with q = +1.0 or 0 maintain their maximum bending strength over a reasonably large range of end rotation. Consequently a good estimate of the total available column strength at a joint is achieved by adding the maximum moments for the two columns as shown for Case~~s~~ 1 ~~and 2~~ of Fig. ~~4.1.~~ 5.1 When q = −1.0 however, the strength varies continuously with end rotation, so the rotations must be considered when evaluating the total available column strength at a joint as shown by Case~~s~~ ~~3~~ 2 ~~and 4 in Art. 4.9.~~ of Fig. 5.1

In most instances, it will not be necessary to consider the interior column rotations because q will be between +1.0 and 0, or in many cases where q is between 0 and −1.0, the column end moments are so small they can be neglected. This is especially true where girder live-load reductions have been used.

In summary, the following procedure is recommended for checking column strength under checkerboard loading:

1. Evaluate the net girder moment at Levels U, C, and L using Eqs. ~~6.1~~ 7.1 and ~~6.2~~ 7.2(a,b); distribute one-half of the moment to the columns above and below each joint; and calculate q. For a bent with repetitive girder framing and loading, one set of calculations will suffice.
2. In some cases it will be sufficient to observe that plastic hinges form at the ends of the beams under factored dead load alone, that is $M_d = M_p$. Then checkerboard loading causes no significant difference from full loading and no further check is required.
3. For q between +1.0 and 0 (all exterior columns and most interior columns):
   a. If $h/r_y < 25$ and $P/P_y < 0.9$, column strength is the same as full loading; if values of h/r_y and P/P_y fall below the curve in Fig. ~~6.2,~~ 7.2 lateral torsional buckling does not govern. Compare the maximum column moment with the allowable moment determined for full loading.

b. Step (a) eliminates most columns from further checks. When the conditions of Step (a) do not apply, the column strength for $q = 0$ may be determined from Design Aids II and III and compared with the applied loads as outlined in Art. ~~4.9~~. 5.8

4. For q between 0 and -1:

a. If the column moments do not exceed $0.05M_{pc}$, the column will be adequate for major-axis bending since under these conditions a small redistribution of the column moments can be accommodated.

b. If the column moments are in excess of $0.05M_{pc}$, use Design Aids II and III as described in Art. ~~4.9~~ to determine the column strength. 5.8

The calculations for column end moment and q for Bents A and B are given in Tab. ~~8.24~~. Since 9.27 and Tab 9.28, respectively plastic hinges form under factored dead load in Bent A, the net girder moment and q for checkerboard loading will be the same as those for the gravity loading. Full gravity loading was checked in Tab. ~~8.8~~; all columns of Bent A are 9.7 and Tab. 9.8 satisfactory. Since $q = 0$ for Bent B, Step 3(a) is used to check the columns in Tab. ~~8.24~~; all the columns are satisfactory.

9.28 ; all the interior columns satisfactory. However, the exterior columns above and below level 4 must be changed. They are replaced by a larger size to satisfy the requirement for LTB.

~~6.4~~ DEFLECTIONS AT WORKING LOAD 7.4

The deflection requirements in Section 1.13 of the AISC Specification will be used as a guide. The live load deflection of the floor girders must be less than 1/360 span.

As a first step in checking deflections, all girders will be assumed simply supported. If the deflection guides are satisfactory for simple supports, then they must also be satisfied for the real girders that have restrained ends. The midspan deflection ratio of a simply supported girder is:

$$\frac{\delta}{L_g} = \frac{5\, w_l L_g^{\,3}}{384\, EI} \qquad \text{~~(6.3)~~} \;(7.3)$$

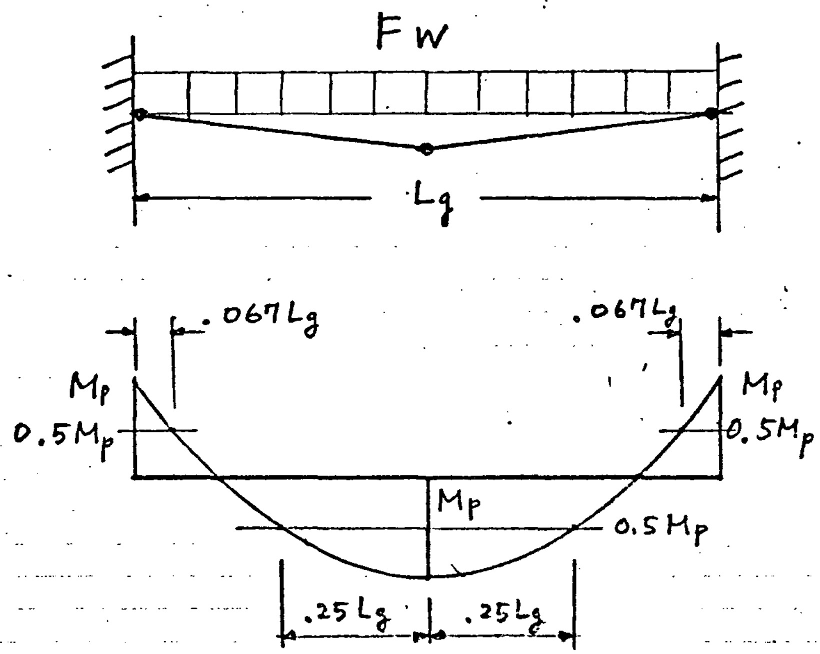
Reduced live loads w_l are used in the calculations, and deflections are calculated only at working load. In Tab. ~~8.25,~~ the live-load deflec- 9.29
tions at service loads are calculated and compared with 1/360 L_g All girders satisfy this requirement. In the check on Bent B, only the they are
lightest ~~girder is~~ considered since ~~it is~~ the most girders among those below level 2 are
critical.

~~6.6~~ SPACING OF LATERAL BRACING 7.5

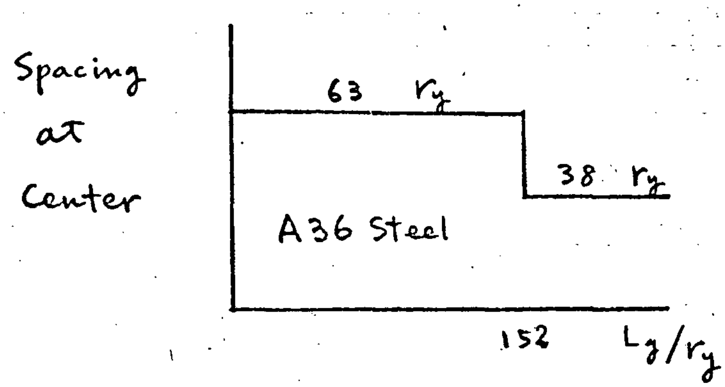
~~In a braced multistory frame the moment diagram at the girder design condition is shown in Fig. 6.4(a).~~ Lateral bracing of the compression flange is required in the vicinity of the plastic hinges to ensure that M_p can be reached and a mechanism can form. The lateral bracing requirements are given in Tab. ~~3.1.~~ 3.1 These requirements can be given in a more convenient design form for a uniformly loaded girder by combining them with the moment diagram. ~~of Fig. 6.4(a).~~

For instance, the moment diagram of an interior girder in supported Bent A is shown as Fig. 7.3(a), then the

~~The~~ required bracing spacing at the plastic hinge locations (center and both ends) is given in Fig. ~~6.4(b)~~ 7.3(b) for A36 steel. These rules were derived by determining the range over which a bracing rule is applicable. For example, from Fig. ~~6.4(a)~~ 7.3(a), at the center ~~$M/M_{pc} > 0.7$~~ $M/M_p > 0.5$ if



(a) Moment Diagram - Uniformly Loaded Beam



(b) Design Aid for Bracing Spacing on A36 Beams

Fig. 7.3 SPACING OF BRACING FOR UNIFORMLY LOADED BEAM

$0.25L_g$

$l_{cr} <$ ~~$0.194L_g$~~, so $l_{cr} =$ ~~$38r_y$~~. Eliminating l_{cr} gives $38r_y$
$L_g/r_y >$ ~~$38/0.194 = 196$~~. At the center if $38/0.25 = 152$
$L_g/r_y >$ ~~196~~, then braces must be spaced at ~~$38r_y$~~. $38r_y$
For $L_g/r_y <$ ~~196~~, a spacing of ~~$65r_y$~~ is permissible. 153 $63r_y$
Thus it is only necessary to calculate L_g/r_y 153
to determine the bracing spacing at the center.

Bracing from the ends can be placed at ~~$65r_y$~~. $63r_y$
For the ~~$38r_y$~~ rule to govern, it would be $38r_y$
necessary to have a girder with a $L_g/r_y >$ ~~825~~. $38/0.0785 = 485$
For rolled sections, such a girder cannot exist because deflection limitations would restrict the girder length to a much smaller value.

The maximum bracing spacing for the compression flange for the girders of Bents A and B are given in Tab. ~~8.26~~. 9.30 and Tab. 9.31, respectively. In Bent B plastic hinges are formed at the leeward ends only for combined gravity and wind loadings, F = 1.3, as shown in the figure. As a practical consideration, bracing can be provided only at the joist locations. A tentative floor system design has established a 3-ft. joist spacing for the exterior bays and a 2-ft. spacing for the interior bay. The joists will be positively attached to the top flange of the girders. Tab. ~~8.26~~ 9.30 and Tab. 9.31 show~~s~~ that in all cases, the allowable bracing spacing is greater than the joist spacing so the top flange is adequately braced. Bracing may also be required in the compression regions of the bottom flange. In Bent A, a short length of the bottom flange is in compression at the ends of the girder. Since the girder is rigidly attached to the column and the length of the negative moment region is less than ~~$65r_y$~~ $63r_y$, no bracing is necessary. ~~In Bent B, however, the exterior bay girders have a compression region at the bottom flange at midspan where the K-brace connection is located. At this point, bottom flange bracing must be provided. This can be accomplished by welding joist chord extensions to the bottom flange of the girder. In summary, the joists will provide adequate top flange bracing for the girders. No other bracing is required for Bent A but the exterior bay girders of Bent B require two bottom flange braces near midspan as shown in Tab. 8.26.~~

~~6.7~~ 7.6 EFFECT OF SHEAR ON BENDING CAPACITY

Eq. ~~3.3~~ 3.5 gives the maximum allowable shear force which a member can resist. If the actual shear is greater, then the web of the section must be strengthened or the member size increased. ~~The shear in the girders is checked in Tab. 8.27~~. The maximum applied shear is given by

$$V_{max} = \frac{FwL_g}{2} \qquad \cancel{(6.12)} \quad (7.4)$$

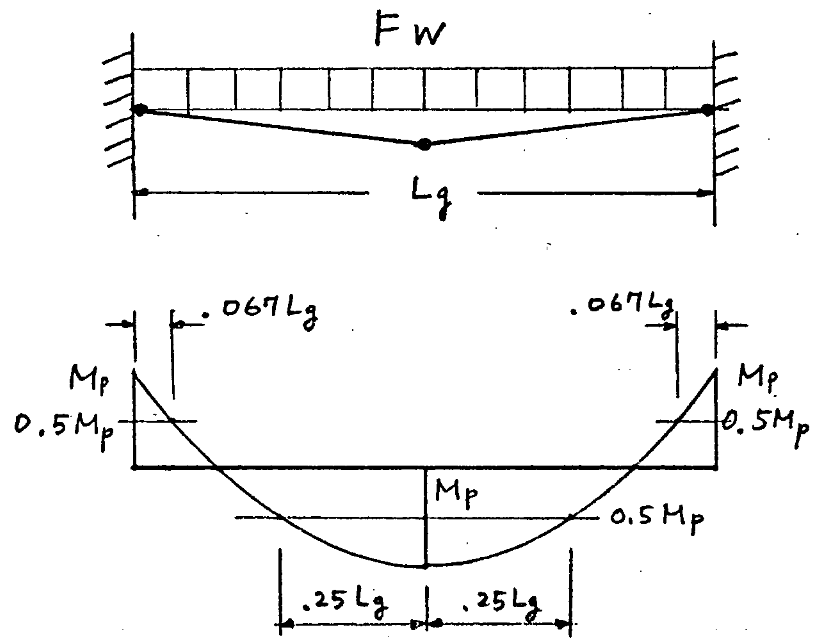
from equilibrium or symmetry. The largest shear occurs when $F = 1.7$ for Bent A ~~All girders for Bents A and B are satisfactory as shown in Tab. 8.27~~. and the shear is checked in Tab. 9.32. On the other hand the maximum shear in the girders of Bent B must be determined by comparing V_{max} in Eq. (7.4) for $F = 1.7$ with the value obtained by Eq. (7.5) for $F = 1.3$ as shown in Tab. 9.33 .

$$V_{max} = \frac{FwL_g}{2} + \frac{M_a + M_b}{L_g} \qquad (7.5)$$

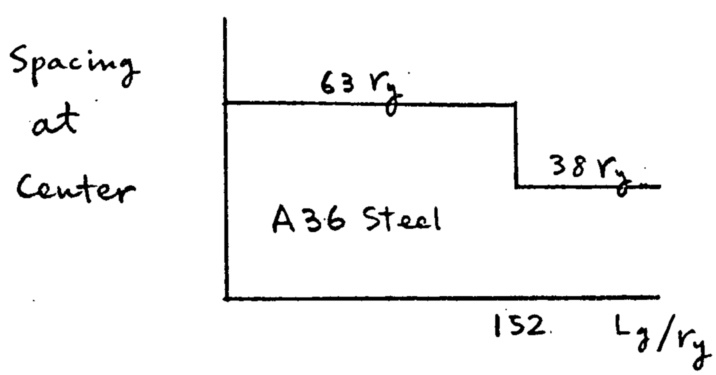
All girders for Bent A and B are satisfactory

~~6.8~~ 7.7 UPLIFT AT FOOTINGS—BENT *B*

The engineer must provide for possible uplift forces at the footings of Bent *B* under combined load. ~~An estimate of these uplift forces is given in Tab. 8.28. At working load wind can cause 203 kips uplift at the exterior footing and 268 kips uplift at the interior footing. The exterior column uplift can be resisted by the exterior foundation wall carrying shears to the adjacent Bents A. Interior column uplift could be accommodated by bracing in the interior bay at the bottom level.~~ In general, uplift forces do not occur in unbraced frames. But in a multistory frame with narrow bays these forces are possible. Tab. 9.19 (5) and Tab. 9.20 (5) show no uplift forces in Bent B.



(a) Moment Diagram – Uniformly Loaded Beam



(b) Design Aid for Bracing Spacing on A36 Beams

Fig. 7.3 SPACING OF BRACING FOR UNIFORMLY LOADED BEAM

CHAPTER ~~7~~ 8

Connections

~~7.1~~ 8.1 INTRODUCTION

The successful performance of every structure depends upon the connections as well as upon the main members. Connections that are not capable of ~~achieving the assumed degree of end fixity cause the girders to carry higher mid span moments than allowed for in design. Thus, the behavior of the structure as a whole is changed and its ultimate strength~~ satisfying the assumptions made in the design cause the behavior of the structure as a whole to be changed and the ultimate strength of each member may be quite different from that computed by the designer.

Design of a connection must consider not only angles, plates, welds and bolts but also the webs and flanges of girders and columns near the juncture.

~~The requirements for connections are:~~

~~1. strength~~
~~2. rigidity~~
~~3. lack of interference with architectural features~~
~~4. economical fabrication~~
~~5. ease of erection~~

The requirements for connections are:

1. strength
2. stiffness
3. rotational capacity (ductility)
4. lack of interference with architectural features
5. economical fabrication
6. each of erection

These are requirements for allowable stress design as well as for plastic design. The performance of connections depends on the ductility of the steel to produce a redistribution of localized stress peaks, and it is the ultimate strength, substantiated by physical tests, that provides the basis for design of connections by either method.

For plastically designed structures, strength and rigidity are important requirements. Connections located at points of maximum moment must not only develop the plastic moment M_p in the connected members, but must maintain these members in their relative positions while plastic hinges develop at other locations.

Phenomena that may affect the development of strength and adequate rotation are:

1. excessive column web shear deformation causing loss of strength
2. column web crippling influencing strength and rotation
3. excessive column flange distortion leading to weld and fastener failures
4. poor welding and poor welding details
5. improper bolt tension

~~7.2~~ 8.2 TYPES OF CONNECTIONS

In multistory building frames the important connections to be considered are: beams to girders, interior tie beams and spandrel beams to columns, girders to columns, column splices, and bracing to girders and columns. Connections are classified according to the AISC designation as:

~~Type 1. "Rigid-frame"—girder-to-column connections have sufficient rigidity to hold virtually unchanged the original angles between intersecting members until M_p develops in a region immediately adjacent to the connection.~~

~~Type 2. "Simple"—assumes ends of beams and girders are connected for shear only and are free to rotate from the beginning of loading.~~

Type 1, commonly designated as "rigid-frame" (continuous frame), assumes that beam-to-column connections have sufficient rigidity to hold virtually unchanged the original angles between intersecting members.

Type 2, commonly designated as "simple" framing (unrestrained, free ended), assumes that, insofar as gravity loading is concerned, ends of beams and girders are connected for shear only, and free to rotate under gravity load.

Type 3, commonly designated as "semi-rigid framing" (partially restrained), assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type 1 and the flexibility of Type 2.

As noted in Art. 4.2 the application of plastic design principles to multistory braced bents requires the use of Type 1 connections between the girders and columns of the Supported Bents A and the Braced Bent B. The connections for the tie beams and spandrels between these bents are Type 2 to avoid introducing biaxial bending into the columns. Beam-to-girder connections may be Type 1 or 2.

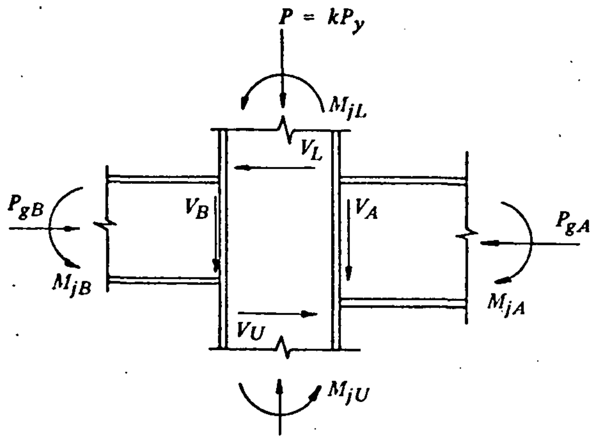
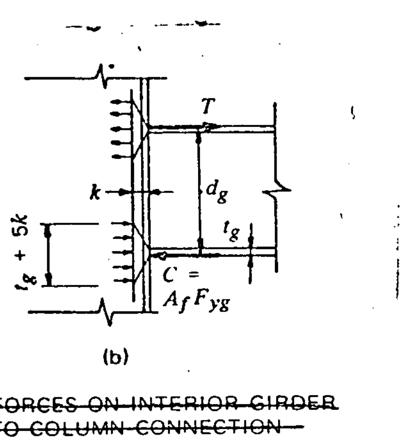
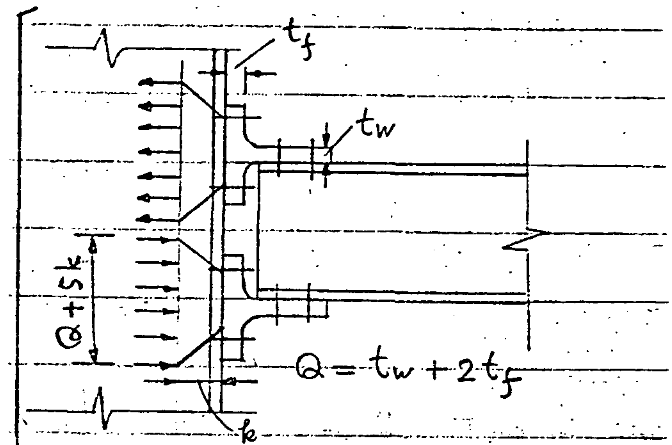


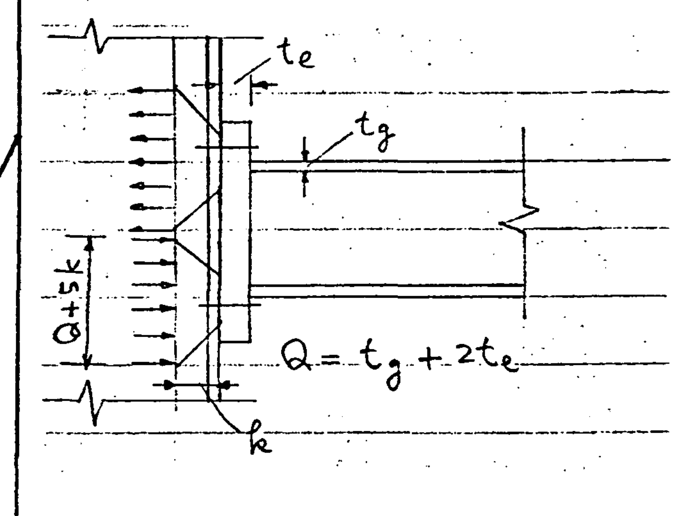
Fig. 8.1 Forces on Interior Girder to Column Connection



(a) Fully welded connection



(b) T-stub connection



(c) End plate connection

Fig. 8.2 Distribution of Flange Forces in Moment Resisting Connections

~~The compression flange force C fans out as it is transmitted through the column flange to the toe of the fillet where it may cripple the column web w_c. Research has shown that crippling will not occur if the following inequality is satisfied:~~

$$\cancel{w_c(t_g + 5k)F_{yc} \geqslant A_f F_{yg} \qquad (7.1)}$$

Stiffening the column should be required to prevent premature failure of a joint component due to column web crippling or column flange deformation. Otherwise, the full capacity of a connection can not be developed.

On the compression side of the beam, crippling of the column web should be avoided. If the compression flange force C is assumed to be distributed over a region Q on the column face and to fan out on a 2.5:1 slope from the point of contact to the k-line of the column web, the force in the beam flange may be resisted by a length at the k-line of column web equal to (Q + 5k), where Q is given as

$Q = t_g$ for the welded connection of Fig. 8.2(a)

$Q = t_w + 2t_f$ for the T-stub connection of Fig. 8.2(b)

$Q = t_g + 2t_e$ for the end plate connection of Fig. 8.2(c)

Crippling will not occur if the force as given in the resisting force is greater than or equal to the flange following inequality

$$W_c(Q+5k)F_{yc} \geq A_f F_{yg} \qquad (8.1)$$

where W_c = thickness of column web
A_f = ares of one girder flange
F_{yc} = yield stress of the column
F_{yg} = yield stress of the girder

For a column with a slender web, for which

$$d_c/w_c > 180/\sqrt{F_{yc}} \qquad (8.2)$$

where d_c = the depth of the column, stability of the compression region may govern rather than strength alone. By considering the post-buckling strength of the column web, it is suggested that the following relationship be satisfied (Ref. 16)

$$W_c^3 \leq \frac{F_{yg}}{F_{yc}} A_f \frac{d_c\sqrt{F_{yc}}}{4100} \qquad (8.3)$$

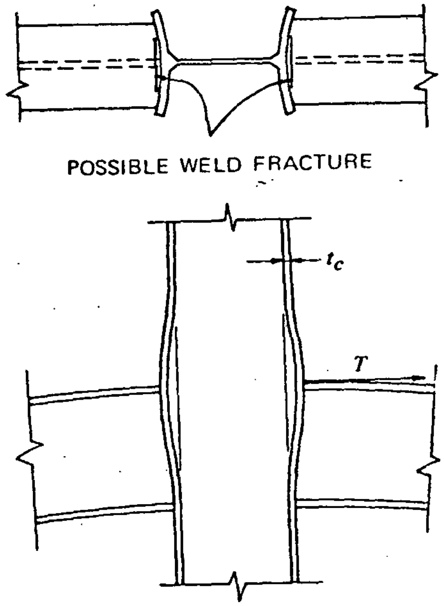


FIG. 7.2 BENDING OF COLUMN FLANGES DUE TO TENSILE FLANGE FORCE

Fig. 8.3

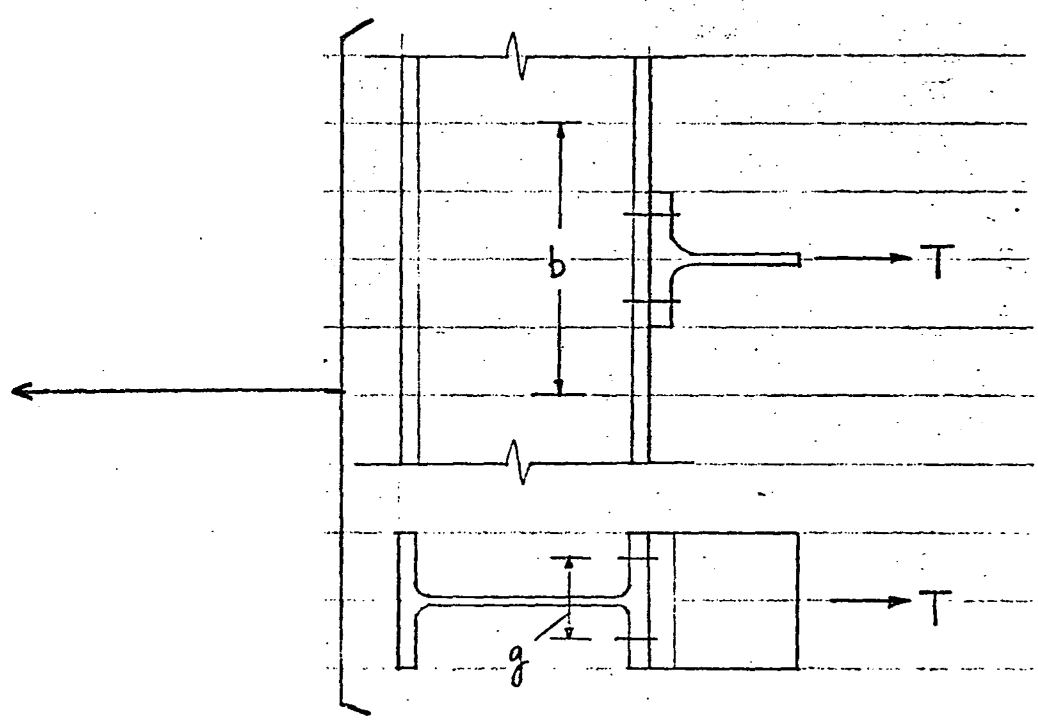


Fig. 8.4 Effective Length of Column Flange

The tensile flange force T has a different effect on the column. It bends the column flange as shown in Fig. ~~7.2~~ 8.3 in a welded connection and in the process the ductility of the weld joining the girder flange to the column may be exceeded, causing weld fracture. Research has shown that this is not likely to occur if the column flange thickness satisfies the following inequality:

$$t_c \geqslant 0.4 \sqrt{A_f \frac{F_{yg}}{F_{yc}}} \qquad \cancel{(7.2)}\ 8.4$$

If a T-stub or an end plate is bolted to the column flange, the connected tension force is distributed into the column flange through the fasteners. Therefore, the need for column stiffening should be estimated differently. A column flange of an effective length, b, can be considered to resist the bending moment caused by the tension force, T/2, at a distance of g/2 from the center line of the web as shown in Fig. 8.4. If the following inequality satisfied the plastic bending strength along the fillet of column section will exceed the bending moment along the fillet

$$\left(\frac{T}{2}\right)\left(\frac{g}{2}\right) \leq \frac{bt_c^2}{4} F_{yc} \qquad (8.5)$$

where T is the applied force and g the fastener gage. The effective length, b, is defined as

$$b = r + \frac{3g}{2} \qquad (8.6)$$

where r is the fastener pitch or spacing along the column flange (Ref. 17).

If the requirements of Eqs. ~~7.1 and 7.2~~ 8.1 to 8.5 are not satisfied, additional resistance must be provided by stiffeners welded between the column flanges, either horizontally in line with the girder flanges or vertically between the column flange tips as shown in Fig. ~~7.3~~ 8.5. Vertical stiffeners are considered to be only 50% as effective as horizontal stiffeners. The following equations are used to proportion stiffeners arranged in symmetrical pairs.

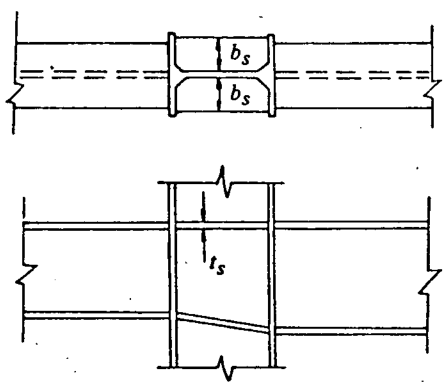
Horizontal stiffeners:

$$A_f F_{yg} - \cancel{w_c(t_g + 5k)F_{yc}}\ W_c(Q+5k)F_{yc} - 2b_s t_s F_{ys} = 0 \quad \cancel{(7.3)} \qquad (8.6)$$

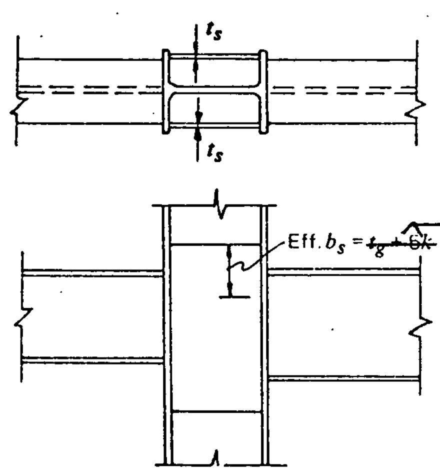
Vertical stiffeners:

$$A_f F_{yg} - \cancel{w_c(t_g + 5k)F_{yc}}\ W_c(Q+5k)F_{yc} - \cancel{\tfrac{1}{2} \times 2(t_g + 5k)t_s F_{ys}}\ \tfrac{1}{2} \times 2(Q+5k)t_s F_{ys} = 0 \quad \cancel{(7.4)} \qquad (8.7)$$

where t_s = thickness of the stiffener
F_{ys} = yield stress of stiffener
Q = effective thickness as defined in Fig. 8.2



(a) HORIZONTAL STIFFENERS



(b) VERTICAL STIFFENERS

FIG. ~~7.3~~ 8.5 TYPES OF COLUMN STIFFENERS

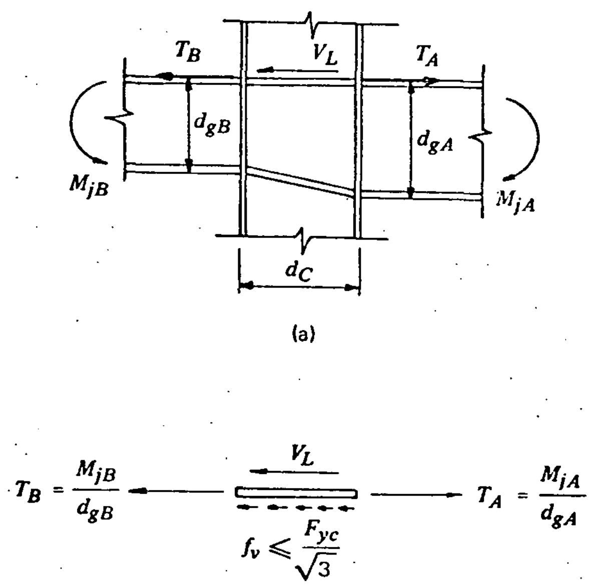
An unbalance of girder moments at a girder-to-column connection produces shear in the column web. If the shear stress in the web is excessive, diagonal stiffeners or a doubler plate must be used. The forces on an interior connection are shown in Fig. ~~7.4a,~~ 8.6a where M_{jA} is greater than M_{jB} and V_L is the shear in the column just above the top stiffener. Fig. ~~7.4b~~ 8.6b shows a freebody diagram of the top stiffener. Column web shearing stresses are required for equilibrium. Assuming that the shearing yield stress is $\frac{F_y}{\sqrt{3}}$ the following inequality must be satisfied:

$$w_c\, d_c \frac{F_{yc}}{\sqrt{3}} \geqslant T_A - T_B - V_L \qquad \cancel{(7.5)}\ (8.8)$$

If the thickness of the column web is less than that required by Eq. ~~7.5,~~ 8.8 diagonal stiffeners or doubler plates must carry the excess shear.

At a girder-to-column connection in a lower story of an unbraced frame, the bending moments of girders may be antisymmetrical as shown in Fig. 8.7. Then, the following inequality must be satisfied instead of Eq. (8.8).

$$W_c d_c \frac{F_{yc}}{\sqrt{3}} \geq T_A + T_B - V_L \qquad (8.9)$$



(a)

(b)

FIG. ~~7.4~~ 8.6 SHEAR STRESS IN COLUMN WEB

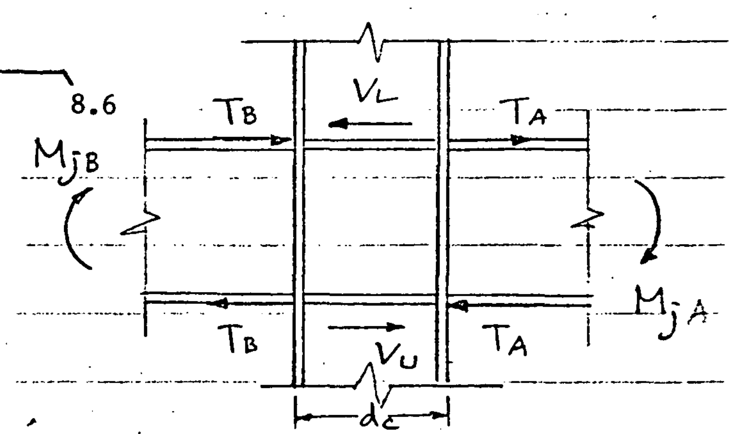


Fig. 8.7 Shear Stress in Column Web Due to Antisymmetrical Bending

The design of diagonal stiffeners is based on the stiffener carrying the excess shear. Thus, from Fig. ~~7.5,~~ the required area of two stiffeners symmetrically arranged is given by: 8.8

$$A_s F_{ys} \cos\theta \geqslant T_A - T_B - V_L - w_c d_c \frac{F_{yc}}{\sqrt{3}} \quad \cancel{(7.6)} \qquad (8.10)$$

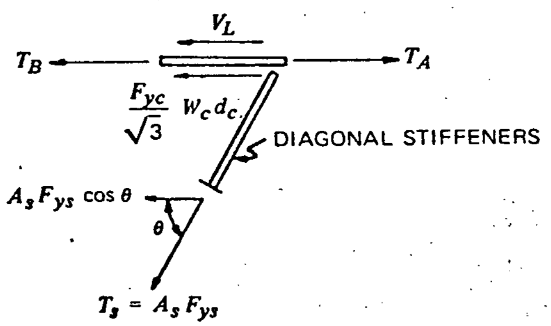


FIG. ~~7.5~~ FORCES ON DIAGONAL STIFFENERS 8.8

Corner and exterior connections are special cases of the condition described above and similar analyses hold.

~~The effect of high axial stress on the shearing resistance of the column web is a subject of continuing research, but it is believed to be of only academic interest; beam-columns with high axial load allow only a small percentage of the strength to carry moments that produce column web shear.~~

The effect of high axial stress on the yielding of column webs due to shear has been studied in the experimental research. However, post-yield strain-hardening counteracts the deterioration of shear strength sufficiently to make it primarily a matter of academic interest.

~~7.4~~ WELDED CONNECTIONS 8.4

The welding of girders to columns and of column stiffeners requires welds proportioned by plastic design stress values. Butt welds may be assumed capable of developing on their minimum throat section the tensile yield stress F_y of the base material. Fillet welds may be designed for the shearing yield stress of the weld metal on the minimum throat section. ~~A safe value for design may be obtained by multiplying the allowable working stress value by 1.67. Thus for E60 electrodes,~~

$$\cancel{F_y = 1.67 \times 13.6 = 22.7 \text{ ksi}}$$

A value for design may be obtained by multiplying the allowable working stress value by 1.70. Thus for E70 electrodes,

$$F_y = 1.70 \times 21.0 = 35.7 \text{ ksi}$$

7.5 BOLTED CONNECTIONS

It is economical to shop weld as many parts of a connection as possible. However, the field connection may be accomplished most economically by welding or high strength bolting, depending on such factors as local codes, availability of labor, or the inspection procedures required.

Since the allowable stress design of bolted connections is based upon their behavior at ultimate load, the design of bolted connections for a plastically designed structure involves similar procedures, except that the ultimate strength of the bolts must be used instead of allowable stress.

In plastic design, as in allowable stress design, the designer should be free to decide which bolted connections must be friction-type and which may be bearing-type. Connections subjected to stress reversal or where slippage would be undesirable must be friction-type. Thus, girder moment connections and bracing connections subjected to wind reversal should be designed as friction-type, but girder shear connections ~~might~~ can be bearing-type. ~~However, the AISC Specification states in Section 2.7, "when used to transmit shear produced by the ultimate loading, one bolt may be substituted for a rivet of the same nominal diameter". This amounts to recognition of only friction-type connections in plastically designed structures.~~

(deletion)

~~The allowable "shear" stresses prescribed for high strength bolts in friction-type connections give a factor of safety against slip of about 1.4 under working gravity loads. When the shear stress is increased one third for wind, the factor of safety approaches unity. Thus, when the allowable stresses are multiplied by 1.67 to obtain an ultimate shear stress, slip will occur under all factored loading conditions. Of course, it is not expected that factored loading will actually act on the structure.~~ (deletion)

High strength bolts that resist tension resulting from factored loading may be designed for resisting a tensile force equal to the guaranteed minimum proof load. Thus, even under factored loading it is unlikely that the initial installation tension will be exceeded. In calculating the applied tensile force on a bolt, allowance should be made for tension caused by prying action.

7.6 COLUMN SPLICES

Column sections change and are spliced every second or third story. The splice is usually placed about two feet above the floor level. The splice must be designed for:

1. An axial compressive force resulting from the factored dead and live load. ($F.$=1.7)
2. Axial compression force plus shear and moment caused by wind acting in conjunction with dead and live load. ($F.$ = 1.3)
3. Axial tensile force plus shear and moment when tension occurs under a condition of full factored wind load combined with 75% of the factored dead load, and no live load. ($F.$ = 1.3)

According to the AISC Specification, in tier buildings 100% of the axial compression force may be transmitted from one column section to the next by bearing, provided that both sections are milled. Partial penetration groove welds having no root opening may be used to join column flanges when the stress to be transferred will permit them.

When columns of the same nominal depth are spliced, full bearing is possible because the inside-of-flange dimension is the same for all weights. The weld or bolts and the splice material serve only to hold all parts securely in place. If the lower column is much deeper than the upper one, it is necessary to weld stiffeners on the inside of the lower column flange to provide an adequate bearing surface. Alternative solutions are to provide a bearing butt plate on the lower column or to develop the strength of fills fastened on the outside of the flanges of the upper column.

Horizontal shear forces are resisted by plates on both sides of the column webs extending across the joint of the upper and lower column sections. If a butt plate is used, shear is resisted by bolts connecting web angles to the butt plate. Web plates or angles also aid erection by holding the column sections in line during field welding.

Tension resulting from significant moments at column splices is transmitted by full penetration flange welds or by splice plates fillet welded or bolted to the flanges. For typical details see Ref. 9.

~~7.7 BRACING CONNECTIONS~~

~~Diagonal bracing is often laid out with its centerline intersecting the centerlines of girders and columns as for a pin connected truss. This arrangement usually permits the horizontal component of the bracing force to be transmitted into the girder flange and the vertical component into the column flange—a direct transfer into the logical resisting member without introducing a shear into the other. However, other considerations often cause deviations from this ideal arrangement. Welded girder-to-column connections, because of their simplicity of detail, facilitate the connecting of bracing.~~

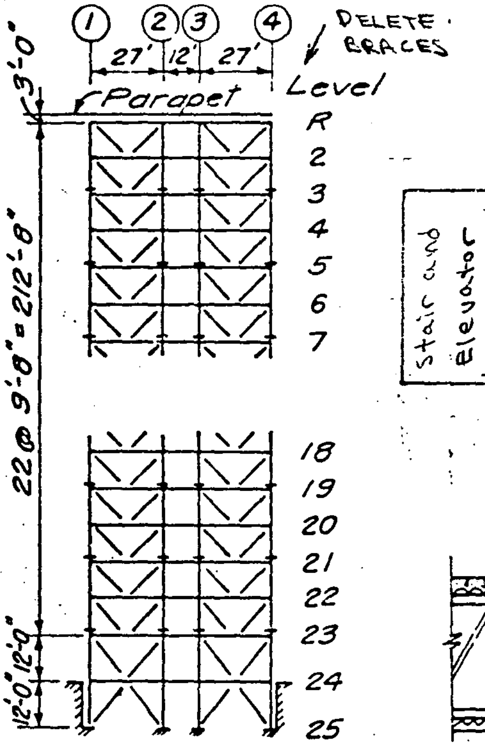
~~Bracing connection details depend upon the type of member used for the bracing, i.e., rods, pairs of angles, H-section, or tubes like the pipe used in the design example. Gusseted connections consisting of plates and angles, or tees shop welded to the brace may be used. The high strength bolt is ideally suited for making the field connection because of its ability to pull up the draw in the brace. Tubes may be connected to gusset plates by slotting the tube, and fillet welding the tube to the plate, or full penetration butt welds joining tubes to end plates provide an excellent and simple connection.~~

(deletion)

~~In K-bracing two diagonals join one another at midspan of the girder. Research[10] has shown that a stronger connection is developed if the centerlines of the pipe braces intersect before reaching the girder centerline, i.e., have a negative eccentricity. This geometric arrangement causes a partial intersection of the pipes and a more direct balancing of the vertical components of the bracing forces.~~

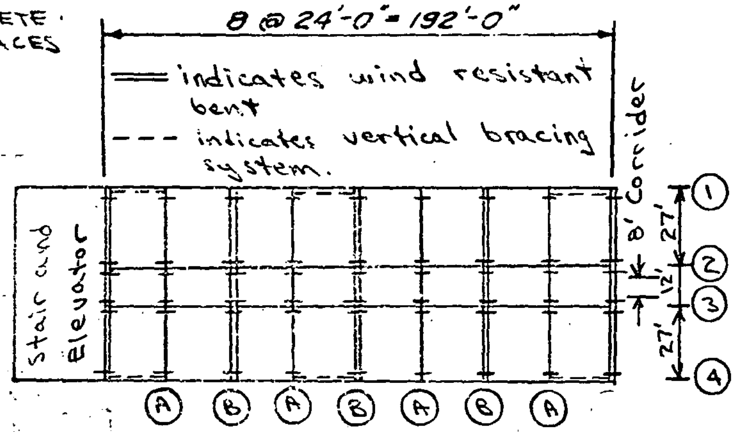
DESIGN EXAMPLE APARTMENT HOUSE TABLE OF CONTENTS	TABLE 9.1

DESIGN EXAMPLE APARTMENT HOUSE PRELIMINARY DESIGN DATA	FIGURE 9.8.1

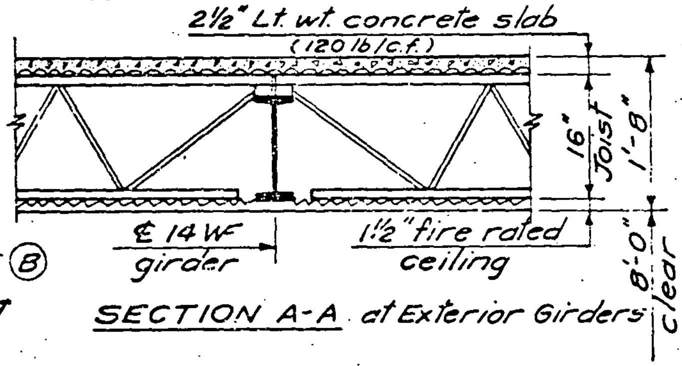


ELEVATION - BENTS (A) & (B)

~~K-bracing for Bent B only~~



FLOOR FRAMING PLAN



SECTION A-A at Exterior Girders

LOADS

Live load reduction per American Std. Bldg. Code A58.1-1955, Sect. 3.5

Floor loads

	Ext. bay	Int. bay
2½" Lt. wt. slab	25	25
Floor finish	1	1
Ceiling	5	5
Partitions	20	40 (1)
Joist	3	4
Mechanical	1	5
Dead load	55 psf	80 psf
Live load	40 psf	60 psf
Total load	95 psf	140 psf

Roof loads

Metal deck	4
Lt. wt. fill	22
Roofing	5
Insulation	2
Ceiling	5
Joist	2
Mechanical	5
Dead load	45 psf
Live load	30 psf
Total load	75 psf

(1) 2 corridor walls at 30 psf × 8'/12' = 40 psf

Exterior walls (average) 62 psf × 9.67' = 600 lb/ft	
Parapet	250 lb/ft
~~2 interior partitions at K-braced bays~~	~~50 psf~~
Wind - full height	20 psf
DL - Column steel + fireproofing 210 plf × 9.67' = 2.0 kips	

Load Factors -

Gravity F = 1.70

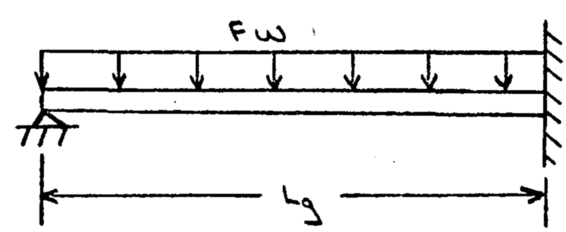
Combined F = 1.30

DESIGN EXAMPLE - PART 1
SUPPORTED BENT A
ROOF GIRDERS

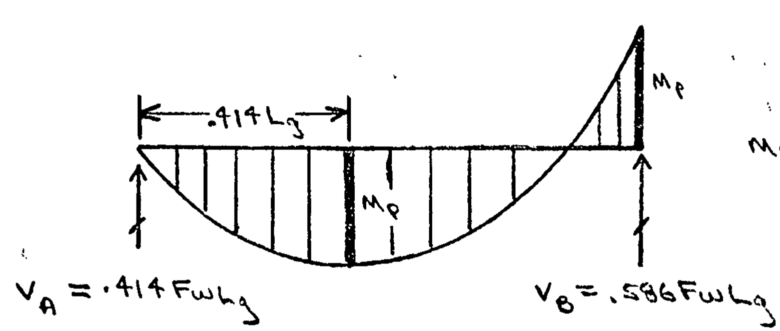
TABLE 9.2

Line	Item	Units	Operation	Bay Exterior	Bay Interior
1	Bay span	ft		27.0	12.0
2	Bent spacing	ft		24.0	24.0
3	Unit DL+LL on roof	psf		75	75
4	Est. column depth	ft		1.0	1.0
5	Clear span	ft	(1)-(4)	26.0	11.0
6	Roof load on girder	Klf	(2)x(3)	1.80	1.80
7	Est. DL of girder	Klf		0.04	0.02
8	Working load	Klf	(6)+(7)	1.84	1.82
9	Factored load	klf	(8)x1.7	3.13	3.09
10	Req'd Mp (1 end simple)	K-ft	$(9)\times(5)^2/11.66$	181.5	---
11	Req'd Mp (rigid)	K-ft	$(9)\times(5)^2/16$		23.4
12	Req'd Z (A36 steel)	in^3	Req'd Mp x12/36	60.5	7.8
13	Section			W14x38	W8x13
14	Provide Z	in^3		61.6	11.7

Hinge Formation - Girder with one end fixed, one end simple.



F = 1.7

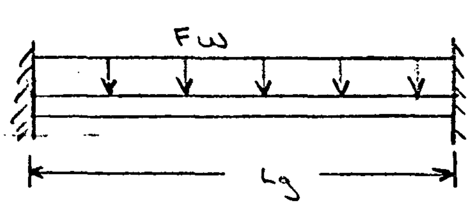


$$M_p = \frac{F w L_g^2}{11.66}$$

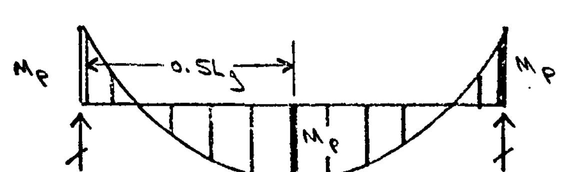
DESIGN EXAMPLE - PART 1 SUPPORTED BENT A FLOOR GIRDERS	TABLE 9.3

Line	Item	Units	Operation	Bay: Exterior	Bay: Interior
1	Bay span	ft		27.0	12.0
2	Bent spacing	ft		24.0	24.0
3	Unit DL on floor	psf		55	80
4	unit LL on floor	psf		40	60
	<u>Live Load Reduction</u>				
5	Floor area	sf	(1) x (2)	648	288
6	0.08 x (floor area)	pct		51.8	23.0
7	$100(D+L)/4.33L$	pct		54.8	53.8
8	Percent LL reduction	pct	Min. (6) or (7)	51.8	23.0
9	Est. column depth	ft		1.0	1.0
10	Clear span	ft	(1) - (9)	26.0	11.0
11	Floor DL on girder	klf	(2) x (3)	1.32	1.92
12	Est. DL of girder	klf		0.04	0.02
13	Reduced LL on girder	klf	$(2)\times(4)\times\left[1-\frac{(8)}{100}\right]$	0.46	1.11
14	working load	klf	(11) + (12) + (13)	1.82	3.05
15	Factored load	klf	(14) x 1.7	3.09	5.19
16	Req'd Mp (1 end simple)	k-ft	$(15)\times(10)^2/11.66$	179.1	
17	Req'd Mp (rigid)	k-ft	$(15)\times(10)^2/16$		39.2
18	Req'd Z (A 36 steel)	in^3	Req'd Mp x 12 / 36	59.7	13.1
19	Section			W 14x38	W 10x15
20	Provide Z	in^3		61.6	16.0

Hinge Formation - Girder with both ends fixed.



F = 1.7



$$M_p = \frac{FwL_g^2}{16}$$

$$V_A = \frac{FwL_g}{2} \qquad V_B = \frac{FwL_g}{2}$$

NOTE: The estimated dead loads of the girders (=0.03) may need fixing. Also, the distribution is not correct.

$.414 \times 27 = 11.2$

$.586 \times 27 = 15.8$

$12 \times 0.5 = 6$

$15.8 + 6 = 21.8$

$11.2 \times .03 = \boxed{.34}$

$21.8 \times .03 = \boxed{.65}$

DESIGN EXAMPLE - PART 1 SUPPORTED BENT A COLUMN LOAD DATA, Working Loads (F=1.0)	TABLE 9.4

Line	Item	Units	Operation	Column	
				Exterior	Interior
	<u>Tributary area per floor</u>				
1	From exterior bay	sf	$27 \times 24 \times \begin{cases} .414 \\ .586 \end{cases}$	268	380
2	From interior bay	sf	$12 \times 24 \times .5$	—	144
3	Total	sf		268	524
4	Unit roof load (DL+LL)	psf		75	75
	<u>Unit floor loads</u>				
5	Exterior bay - dead	psf		55	55
6	- live	↓		40	40
7	Interior bay - dead	↓		—	80
8	- live	↓		—	60
	<u>Loads below roof</u>				
9	DL+LL from roof	Kips	(3)x(4)	20.1	39.3
10	Est. DL girder (@0.03 klf)	↓	$.03 \times \begin{cases} 12 \\ 21 \end{cases}$	0.4	0.6
11	Est. DL column	↓		2.0	2.0
12	DL parapet (@0.25 klf)	↓	0.25 x 24.0	6.0	—
13	working load below roof	↓	Sum (9 to 12)	28.5	41.9
	<u>Loads per floor</u>				
14	DL from floor - Ext. bay	Kips	(1) x (5)	14.7	20.9
15	- Int. bay	↓	(2) x (7)	—	11.5
16	DL girder (@0.03 klf)	↓	$.03 \times \begin{cases} 12 \\ 21 \end{cases}$	0.4	0.6
17	DL ext. wall (@0.60 klf)	↓	0.60 x 24.0	14.4	—
18	DL column	↓		2.0	2.0
19	Total DL per floor	↓	Sum (14 to 18)	31.5	35.0
20	LL from floor - Ext. bay	Kips	(1) x (6)	10.7	15.2
21	- Int. bay	↓	(2) x (8)	—	8.6
22	Total LL per floor	↓	(20)+(21)	10.7	23.8
	<u>Live load reduction</u>				
23	Max. R = 100(D+L)/4.33L < 60	pct	D=(19), L=(22)	~~91.1~~	(1) 57.1
		↓	Limit	60.0	60.0
24	0.08 (trib area) - Level 2	↓	0.08 x (3)	21.4	41.9
25	- Level 3	↓	2 x (24)	42.8	~~83.8~~
		↓	Limit max. R		60.0
26	- Level 4 & below	↓	3 x (24)	~~64.2~~	
			Limit max. R	60.0	60.0
27	<u>Red. LL from floors</u> - below Level 2	Kips	(22) x [1 - R/100]	8.4	13.8
28	- below Level 3	↓	2x(22) x [1 - R/100]	12.2	19.0
29	- below Level 4	↓	3x(22) x [1 - R/100]	12.8	28.6
30	<u>Red. LL increment</u> - Levels 5 to 24	↓	(22) x [1 - 0.60]	4.3	9.5

Note (1) Use 60.0, roof contributes dead load.

DESIGN EXAMPLE - PART I SUPPORTED BENT A COLUMN GRAVITY LOADS	TABLE 9.5

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Exterior Columns					Interior Columns				
Level	DL Kips	Red. LL Kips	Working Load Kips	WLx1.7 Kips	WLx1.3 Kips	DL Kips	Red. LL Kips	Working Load Kips	WLx1.7 Kips	WLx1.3 Kips
Load Increment	31.5	4.3	35.8	60.9	46.5	35.0	9.5	44.5	75.7	57.9
R	21	8	29	49		26	16	42	71	
2	53	16	69	117		61	30	91	155	
3		20	104	177			35	131	223	
4		21	137	233			45	176	299	
5		25	172	292			55	221	376	
6			208	354				266	452	
7			244	415				310	527	
8			279	474				355	604	
9			315	536				399	678	
10			351	597				444	755	
11			387	658				488	830	
12			423	719				533	906	
13			458	779				577	981	
14			494	840				622	1057	
15	Add load increment	Add load increment	530	901		Add load increment	Add load increment	666	1132	
16			566	962				711	1209	
17			602	1023				755	1284	
18			637	1083				800	1360	
19			673	1144				844	1435	
20			709	1205				889	1511	
21			745	1267				933	1586	
22	683		780	1326				978	1663	
23	(1) 715		818	1391				1022	1737	
24	(2) 751	107	858	1459		831	235	1066	1812	

Note (1) DL increment below Level 23
Add DL column 0.21 klf × (12.0 − 9.67) = 0.5 kip

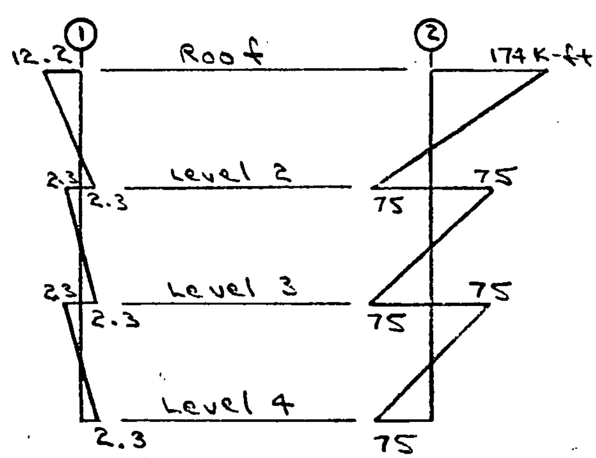
Note (2) DL increment below Level 24
Add DL column = 0.5
Add DL exterior wall 14.4 × $\frac{12.0-9.67}{9.67}$ = 3.5
Tab 9.4(17)
Add 4.0 kips

DESIGN EXAMPLE - PART I SUPPORTED BENT A COLUMN MOMENTS, Factored Gravity Load (F = 1.7)	TABLE 9.6

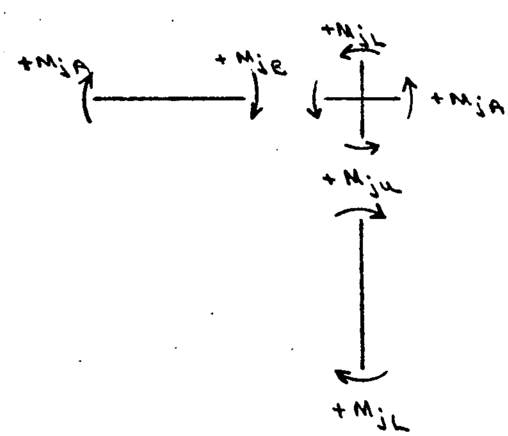
Line	Item	Units	Operation	Column	
				Exterior	Interior
	Moments at roof				
1	Girder left: M_B	K-ft	Tab. 9.2(10)	—	181.5
2	Girder left: V_B	Kips	Tab. 9.2	—	47.7
3	Est. d_c	ft	Tab. 9.2(4)	1.0	1.0
4	At ℄ col. $M_{jB} = M_B + V_B d_c/2$	K-ft	(1) + (2) × (3)/2	—	205.4
5	Girder right: M_A	K-ft	Tab. 9.2(11)	(1) 0	-23.4
6	Girder right: V_A	Kips	Tab. 9.2	33.7	17.0
7	At ℄ col. $M_{jA} = M_A - V_A d_c/2$	K-ft	(5) - (6)(3)/2	-16.9	-31.9
8	Spandrel (6.0K × 0.5 ft × 1.7)	K-ft	Tab. 9.4(12) × $\frac{d_c}{2}$ × 1.7	+5.1	—
9	Column moment at roof	K-ft	-[(4) + (7) + (8)]	+11.8	-173.5
	Moments at Levels 2 to 24				
10	Girder left: M_B	K-ft	Tab. 9.3(16)	—	179.1
11	Girder left: V_B	Kips	Tab. 9.3	—	47.1
12	Est. d_c	ft	Tab. 9.3(9)	1.0	1.0
13	At ℄ col. $M_{jB} = M_B + V_B d_c/2$	K-ft	(10) + (11)(12)/2	—	202.7
14	Girder right: M_A	K-ft	Tab. 9.3(17)	(1) 0	-39.2
15	Girder right: V_A	Kips	Tab. 9.3	33.3	28.5
16	At ℄ col. $M_{jA} = M_A - V_A d_c/2$	K-ft	(14) - (15)(12)/2	-16.7	-53.5
17	Spandrel (14.4K + 0.5 ft × 1.7)	K-ft	Tab. 9.4(17) × $\frac{d_c}{2}$ × 1.7	+12.2	—
18	Net girder moment on joint	K-ft	-[(13) + (16) + (17)]	+4.5	-149.2
19	Column moment	K-ft	0.5 × (18)	+2.3	-74.6

Note (1) $M_A = 0$ due to hinged connection

Column Moment Diagram



Sign Convention



DESIGN EXAMPLE – PART 1
SUPPORTED BENT A
EXTERIOR COLUMNS, FACTORED GRAVITY LOAD (F=1.7)

TABLE 9.7

Below Level	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Req'd P Kips	2.1x M/d Kips	Req'd P_y Kips (a)	h ft.	P_y Kips	r_x in	$\frac{h}{r_x}$	$\frac{P}{P_y}$	Allow M/M_{pc}	Remarks
	Req'd M Kip-ft	Est. d ft.	Trial W Shape	Ratio β	M_p Kip-ft.	r_y in	$\frac{h}{r_y}$	$\frac{M_{pc}}{M_p}$	Allow M Kip-ft	
	Tab. 9.5(4)	2.1x $\frac{(1)}{(2)}$	(1)+(2) or 1.33x(2)	Fig. 9.1	DA-I	DA-I	$\frac{12x(4)}{(6)}$	(1)/(5)	DA-III	
	Tab. 9.6		DA-I	Tab. 9.6	DA-I	DA-I	$\frac{12x(4)}{(6)}$	1.18 x $(1-P/P_y)$	(5)x(8)x(9)	
			A36 STEEL							
R	49 12.2	26 1.0	75 18x17	9.67 +.18	180 47.4	3.36 1.16	35 100	.27 .86	.83 33.6	(LTB) >12.2 OK
2	117 2.3	4.8 1.0	156* ~~8x17~~	9.67 +1.0	180 47.4	3.36 1.16	35 100	.65 .41	0 0	(LTB) <2.3 NG
			8x20		212 57.3	3.43 1.20	34 97	.55 .53	.39 11.8	(LTB) >2.3 OK
4	233 2.3	4.8 1.0	310* 8x35	9.67 +1.0	371 104	3.50 2.03	33 57	.63 .44	1.0 45.6	>2.3 OK
8	474 2.3	4.8 1.0	630* 10x60	9.67 +1.0	636 225	4.41 2.57	26 45	.75 .30	1.0 67.6	>2.3 OK
12	719 2.3	4.8 1.0	956* 12x92	9.67 +1.0	974 421	5.40 3.08	21 38	.74 .31	1.0 130	>2.3 OK
16	962 2.3	4.8 1.0	1279* 12x133	9.67 +1.0	1408 629	5.59 3.16	21 37	.68 .37	1.0 235	>2.3 OK
20	1205 2.3	4.8 1.0	1603* 12x161	9.67 +1.0	1706 778	5.70 3.20	20 36	.71 .35	1.0 270	>2.3 OK
24	1459 2.3	4.1 1.17	1940* 12x190	12.0 0.0	2011 935	5.82 3.25	25 44	.73 .32	1.0 303	>2.3 OK
Alternate design using		F_y = 50 ksi		A572	steel		$\frac{h}{r}$ x1.18			$\sqrt{\frac{50}{36}}$ = 1.18
12	719 2.3	4.8 1.0	956* 10x66	9.67 +1.0	970 345	4.44 2.58	31 53	.74 .31	1.0 105	>2.3 OK
16	962 2.3	4.8 1.0	1279* 12x92	9.67 +1.0	1353 584	5.40 3.08	25 44	.71 .34	1.0 199	>2.3 OK
20	1205 2.3	4.8 1.0	1603* 12x120	9.67 +1.0	1766 777	5.51 3.13	25 44	.68 .37	1.0 291	>2.3 OK
24	1459 2.3	4.1 1.17	1940* 12x133	12.0 0.0	1956 874	5.59 3.16	30 54	.75 .30	1.0 262	>2.3 OK

(a) An * indicates the required P_y is controlled by the condition P/P_y < 0.75

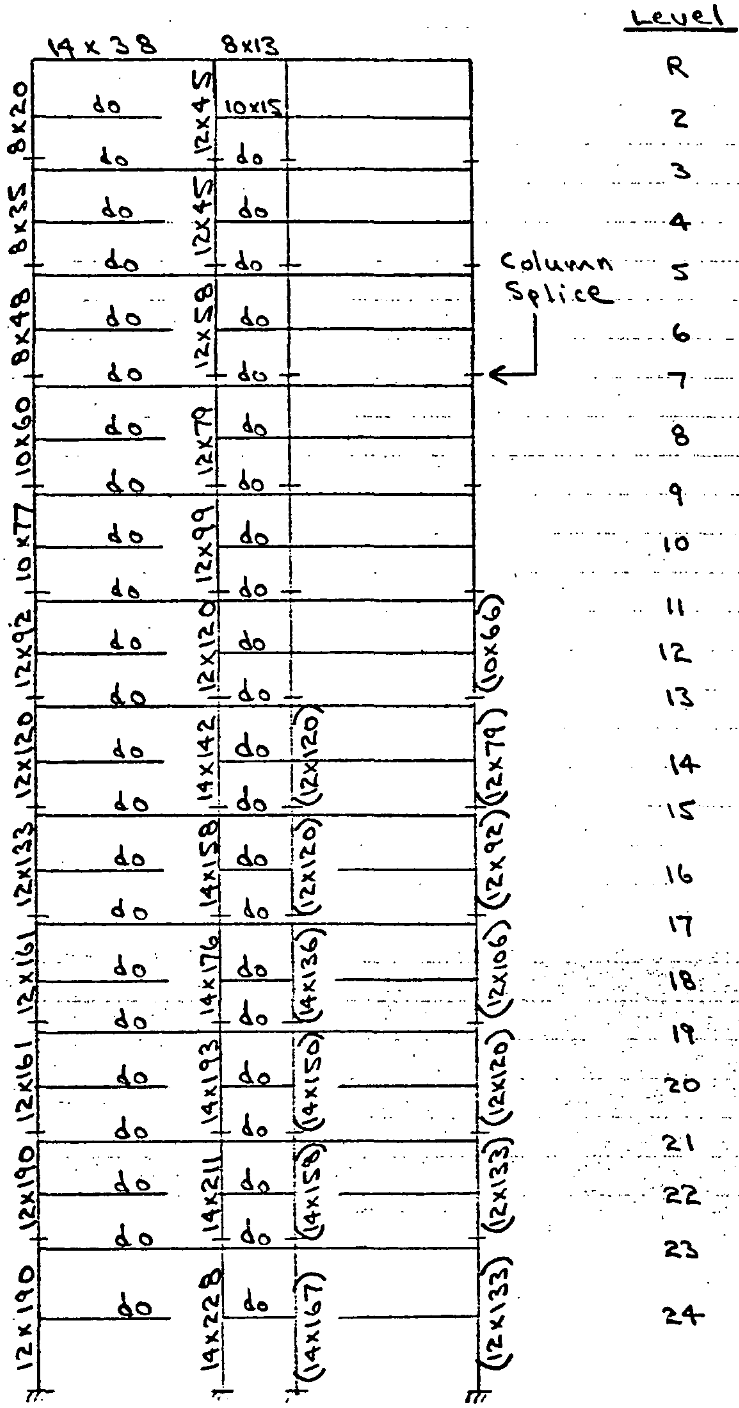
DESIGN EXAMPLE – PART 1
SUPPORTED BENT A
INTERIOR COLUMNS, FACTORED GRAVITY LOAD ($F = 1.7$)

TABLE 9.8

Below Level	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Req'd P Kips	2.1x M/d Kips	Req'd P_y Kips (a)	h ft.	P_y Kips	r_x in	h/r_x	P/P_y	Allow M/M_{pc}	Remarks
	Req'd M Kip-ft	Est. d ft.	Trial W Shape	Ratio q	M_p Kip-ft.	r_y in	h/r_y	M_{pc}/M_p	Allow M Kip-ft	
	Tab. 9.5(9)	2.1x (1)/(2)	(1)+(2) or 1.33x(2)	Fig. 9.1	DA-I	DA-I	12x(4)/(6)	(1)/(5)	DA-III	
	Tab. 9.6		DA-I	Tab. 9.6	DA-I	DA-I	12x(4)/(6)	1.18 x (1-P/P_y)	(5)x(8)x(9)	
	Design using A36 steel F_y = 36 Ksi									
R	71	365	436	9.67	477	5.15	23	.15	1.0	
	174	1.0	12x45	+.43	195	1.94	60	1.0	195	>174 OK
					424	5.13	23	.17	1.0	
			~~12x40~~		173	1.94	60	.98	170	<174 NG
2	155	158	313	9.67	477	5.15	23	.33	1.0	
	75	1.0	12x45	+1.0	195	1.94	60	.80	155	>75 OK
4	299	158	457	9.67	477	5.15	23	.65	1.0	
	75	1.0	12x45	+1.0	195	1.94	60	.41	80	>75 OK
8	604	158	803*	9.67	836	5.34	22	.72	1.0	
	75	1.0	12x79	+1.0	358	3.05	38	.33	117	>75 OK
12	906	158	1205*	9.67	1271	5.51	21	.71	1.0	
	75	1.0	12x120	+1.0	559	3.13	37	.34	189	>75 OK
16	1209	158	1608*	9.67	1673	6.40	18	.72	1.0	
	75	1.0	14x158	+1.0	859	4.00	29	.33	281	>75 OK
20	1511	158	2010*	9.67	2042	6.51	18	.74	1.0	
	75	1.0	14x193	+1.0	1065	4.05	29	.31	327	>75 OK
24	1812	135	2410*	12.0	2414	6.62	22	.75	1.0	
	75	1.17	14x228	0.0	1282	4.10	35	.29	377	>75 OK
	Alternate design using A572 steel F_y = 50 Ksi						$\frac{h}{r}$ x1.18			$\sqrt{\frac{50}{36}} = 1.18$
16	1209	158	1608*	9.67	1766	5.51	25	.68	1.0	
	75	1.0	12x120	+1.0	777	3.13	44	.37	289	>75 OK
20	1511	158	2010*	9.67	2204	6.37	22	.69	1.0	
	75	1.0	14x150	+1.0	1125	3.99	34	.37	417	>75 OK
24	1812	135	2410*	12.0	2454	6.42	26	.74	1.0	
	75	1.17	14x167	0.0	1262	4.01	42	.31	389	>75 OK

(a) an * indicates the required P_y is controlled by the condition $\frac{P}{P_y} \le 0.75$

DESIGN EXAMPLE - PART 1 SUPPORTED BENT A PRELIMINARY MEMBER SIZES	FIGURE 9.2



NOTES:

1) All sections are W-shapes.

(2) All steel is A36, except columns shown as (12x120), which are A572 steel, F_y = 50 ksi.

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B GRAVITY AND WIND LOADS	TABLE 9.9

Line	Item	Units	Operation	Bay: Exterior	Bay: Interior
	<u>Roof Girders</u>				
1	Working load	klf	Tab. 9.2(8)	1.84	1.82
	<u>Floor Girders</u>				
2	Floor DL on girder	klf	Tab. 9.3(11)	1.32	1.92
3	Est. DL of girder	klf		0.10	0.10
4	Reduced LL on girder	klf	Tab. 9.3(13)	0.46	1.11
5	working load	klf	(2)+(3)+(4)	1.88	3.13

Line	Item	Units	Operation	Value
	<u>Joint Wind Loads</u>			
6	Wind pressure	ksf	Fig. 9.1	.02
7	Bent spacing	ft.	Fig. 9.1	24.0
8	No. of supported bents		Fig. 9.1	2
9	Tributary width	ft.	(7) x (8)	48.0
10	Tributary ht., Level 1	ft.	Fig. 9.1, 3+ 9.67/2	7.83
11	Joint load, Level 1	kips	(6) x (9) x (10)	7.52
12	Tributary ht., levels 2-22	ft.	Fig. 9.1	9.67
13	Joint load, Levels 2-22	Kips	(6) x (9) x (12)	9.28
14	Tributary ht., Level 23	ft.	Fig. 9.1, $\frac{9.67+12}{2}$	10.84
15	Joint load, Level 23		(6) x (9) x (14)	10.41
16	Tributary ht., Level 24	ft.	Fig. 9.1, 12/2	6.0
17	Joint load, Level 24		(6) x (9) x (16)	5.76

DESIGN EXAMPLE- PART 2 WIND RESISTANT BENT B COLUMN WORKING GRAVITY LOAD DATA (F=1.0)	TABLE 9.10

Line	Item	Units	Operation	Column	
				Exterior	Interior
	Tributary area per floor				
1	From exterior bay	sf	27x24x.5	324	324
2	From interior bay	sf	12x24x.5	—	144
3	Total	sf	(1)+(2)	324	468
4	Unit roof load (DL+LL)	psf		75	75
	Unit floor loads				
5	Exterior bay - dead	psf		55	55
6	- live			40	40
7	Interior bay - dead				80
8	- live				60
	Loads below roof				
9	DL+LL from roof	Kips	(3)x(4)	24.3	35.1
10	Est. DL girder (@0.03 klf)			0.4	0.6
11	Est. DL column + fireproofing			2.0	2.0
12	DL parapet (@ 0.25 klf)		0.25x24.0	6.0	—
13	Working load below roof		sum (9 to 12)	32.7	37.7
	Loads per floor				
14	DL from floor - Ext. bay	Kips	(1)x(5)	17.8	17.8
15	- Int. bay		(2)x(7)	—	11.5
16	DL girder (@ 0.10 klf)			1.4	2.0
17	DL exterior wall (@ 0.60 klf)		0.60x24	14.4	—
18	DL column			2.0	2.0
19	Total DL per floor		sum (14 to 18)	35.6	33.3
20	LL from floor - Ext. bay		(1)x(6)	13.0	13.0
21	- Int. bay		(2)x(8)	—	8.6
22	Total LL per floor		(20)+(21)	13.0	21.6
	Live load reduction				
23	Max. R= 100(D+L)/4.33L < 60	pct	D=(19), L=(22)	~~86.3~~	(1) 58.7
			Limit	60.0	60.0
24	0.08(trib. area) - Level 2		0.08 x (3)	25.9	37.4
25	- Level 3		2 x (24)	51.8	~~74.9~~
			Limit Max. R		60.0
26	- Level 4 & below		3 x (24)	~~77.7~~	
			Limit Max. R	60.0	
27	Red. LL from floors - below Lev. 2	Kips	(22)x[1- R/100]	9.6	13.5
28	- below Level 3		2x(22)x[1- R/100]	12.5	17.3
29	- below Level 4		3x(22)x[1-R/100]	15.6	25.9
30	Red. LL increment - Levels 5 to 24		(22)x[1-.60]	5.2	8.7

Note (1) Use 60.0, roof contributes dead load.

DESIGN EXAMPLE – PART 2 WIND RESISTANT BENT B COLUMN GRAVITY LOADS	TABLE 9.11

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	Exterior Columns					Interior Columns				
Level	DL Kips	Red. LL Kips	Working Load Kips	WL×1.7 Kips	WL×1.3 Kips	DL Kips	Red. LL Kips	Working Load Kips	WL×1.7 Kips	WL×1.3 Kips
	35.6	5.2	40.8			33.3	8.7	42.0		
R	23	10	33	56	43	24	14	38	65	49
2	59	20	79	134	103	57	28	85	145	111
3		23	118	201	153		31	121	206	157
4		36	166	282	216		40	164	279	213
5		41	207	352	269		48	205	349	267
6			248	422	322			247	420	321
7			288	490	374			289	491	376
8			329	559	428			331	563	430
9			370	629	481			373	634	485
10			411	699	534			415	706	540
11			452	768	588			457	777	594
12			492	836	640			499	848	649
13	Add load increment	Add load increment	533	906	693	Add load increment	Add load increment	541	920	703
14			574	976	746			583	991	758
15			615	1046	800			625	1063	813
16			656	1115	853			667	1134	867
17			696	1183	905			709	1205	922
18			737	1253	958			751	1277	976
19			778	1323	1011			793	1348	1031
20			819	1392	1065			835	1420	1086
21			860	1462	1118			877	1491	1140
22	771		900	1530	1170			919	1562	1195
23	807		942	1601	1225			961	1634	1249
24	847	140	987	1678	1283	790	213	1003	1705	1304

Note (1) DL increment below Level 23
Add DL column 0.21 klf × (12.0 − 9.67) = 0.5 Kip

Note (2) DL increment below Level 24
Add DL column = 0.5
Add DL exterior wall = 3.5 ← Tab. 9.5
Add 4.0 Kips

DESIGN EXAMPLE – PART 2 WIND RESISTANT BENT B FORCES, COMBINED LOADING (F=1.3)	TABLE 9.12

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Level	Wind Load to Bent B (F=1.0) Kips	Factored Wind Shear (F=1.3) Kips	Factored Gravity Loads: Bent A (F=1.3) Kips	Factored Gravity Loads: Bent B (F=1.3) Kips	Factored $P(\Delta/h)$ Wind Shear, Bent A (F=1.3) Kips	Factored $P(\Delta/h)$ Shear, Bent B (F=1.3) Kips	Total Factored Shear (F=1.3) Kip-ft	Σ Story Col. Joint Moments (F=1.3) Kip-ft	Σ Girder Joint Moments (F=1.3) Kip-ft	Σ Column Joint Moments: Above (F=1.3) Kip-ft	Σ Column Joint Moments: Below (F=1.3) Kip-ft
	Tab 9.9 (11, 13, 15 or 17)	1.3 × Σ(1.)	2 × Tab. 9.5 (5+10)	2× Tab. 9.11 (5+10)	(3) × $(\Delta/h)_{ult}$	(4) × $(\Delta/h)_{ult}$ (a)	(2)+(5)+(6)	H × (7)	Avg. (8), Level (n, n-1)	(8) × DF_1 (b)	(8) × DF_2 (b)
R	7.5	9.8	186	184	0.9	0.9	11.6	112	56	0	-56
2	9.3	21.9	416	428	4.2	4.3	30.4	294	203	-56	-147
3	9.3	34.0	610	620	6.1	6.2	46.3	448	371	-147	-224
4	9.3	46.1	814	858	8.1	8.6	62.8	607	528	-224	-304
5	9.3	58.2	1022	1072	10.2	10.7	79.1	765	686	-304	-383
6	9.3	70.3	1232	1286	12.3	12.9	95.5	923	844	-383	-462
7	9.3	82.3	1440	1500	14.4	15.0	112	1080	1002	-462	-540
8	9.3	94.4	1650	1716	16.5	17.2	128	1238	1159	-540	-619
9	9.3	107	1858	1932	18.6	19.3	145	1401	1320	-619	-701
10	9.3	119	2066	2148	20.7	21.5	161	1558	1480	-701	-779
11	9.3	131	2274	2364	22.7	23.6	177	1714	1636	-779	-857
12	9.3	143	2486	2578	24.9	25.8	194	1872	1793	-857	-936
13	9.3	155	2690	2792	26.9	27.9	210	2028	1950	-936	-1014
14	9.3	167	2902	3008	29.0	30.1	226	2186	2107	-1014	-1093
15	9.3	179	3110	3226	31.1	32.3	242	2343	2265	-1093	-1172
16	9.3	191	3320	3440	33.2	34.4	259	2500	2422	-1172	-1250
17	9.3	203	3530	3654	35.3	36.5	275	2656	2578	-1250	-1328
18	9.3	215	3736	3868	37.4	38.7	291	2814	2735	-1328	-1407
19	9.3	227	3944	4084	39.4	40.8	307	2970	2892	-1407	-1485
20	9.3	240	4156	4302	41.6	43.0	325	3138	3054	-1485	-1569
21	9.3	252	4364	4516	43.6	45.2	341	3294	3216	-1569	-1647
22	9.3	264	4570	4730	45.7	47.3	357	3451	3373	-1647	-1726
23	10.4	278	4784	4948	47.8	49.5	375	4504	3978	-1726	-2252
24	5.8	286	5002	5174	50.0	51.7	388	4652	4578	-2252	-2326
							Sum of foundation moments				2326

(a) $(\Delta/h)_{ult}$ is assumed as .005 for top story, and as .01 for stories 2 → 24.

(b) $DF_1 = DF_2 = +0.5$ (assumed)

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B DESIGN OF GIRDERS	TABLE 9.13

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	Bays 1,3					Bay 2			
Level	Σ Girder Joint Moments in Bays, M_g kip-ft	Sway Moment Coefficient G	Plastic Hinge Moment Ratio, R	Req'd M_p kip-ft	Req'd Z, A36 steel in³	Sway Moment Coefficient G	Plastic Hinge Moment Ratio, R	Req'd M_p kip-ft	Req'd Z, A36 steel in³
	Tab 9.12 (9) × DF⁻¹ Note (a)	$\frac{(1) \times (1-\frac{d_c}{L})}{M_{pm}}$ Note (b)	Note (c)	(3) × M_{pm}	$\frac{12 \times (4)}{36}$	$\frac{(1) \times (1-\frac{d_c}{L})}{M_{pm}}$ Note (b)	Note (c)	(7) × M_{pm}	$\frac{12 \times (8)}{36}$
R	18.7	0.18	1.50	150	50.0	0.97	1.80	31.3	10.4
2	67.7	0.64	1.67	170	56.7	2.05	2.26	67.3	22.4
3	124	1.16	1.87	191	63.7	3.76	3.09	92.1	30.7
4	176	1.65	2.08	212	70.7	5.33	3.97	118	39.3
5	229	2.15	2.30	234	78.0	6.94	4.99	149	49.7
6	281	2.64	2.53	258	86.0	8.51	6.08	181	60.3
7	334	3.14	2.77	282	94.0	10.1	7.21	215	71.7
8	386	3.62	3.02	308	103	11.7	8.35	249	83.0
9	440	4.13	3.29	335	112	13.3	9.50	283	94.3
10	493	4.63	3.56	363	121	14.9	10.6	316	105
11	545	5.12	3.85	392	131	16.5	11.8	352	117
12	598	5.61	4.14	422	141	18.1	12.9	384	128
13	650	6.10	4.44	452	151	19.7	14.1	420	140
14	702	6.59	4.76	485	162	21.3	15.2	453	151
15	755	7.09	5.09	519	173	22.9	16.4	489	163
16	807	7.58	5.42	552	184	24.4	17.4	519	173
17	859	8.07	5.76	587	196	26.0	18.6	554	185
18	912	8.56	6.11	623	208	27.6	19.7	587	196
19	964	9.05	6.46	658	219	29.2	20.8	620	207
20	1018	9.56	6.83	696	232	30.8	22.0	656	219
21	1072	10.1	7.21	735	245	32.5	23.2	691	230
22	1124	10.6	7.57	771	257	34.1	24.3	724	241
23	1326	12.5	8.93	910	303	40.2	28.7	855	285
24	1526	14.3	10.2	1039	346	46.2	33.0	983	328

(a) DF assumed = 1/3 for all bays

(b) $M_{pm} = \frac{1}{16} F_2 w (L-d_c)^2$; $F_2 = 1.30$; $d_c = 14''$

Bays 1,3: $M_{pm} = \begin{cases} 99.8 \text{ k-ft, roof} \\ 101.9 \text{ k-ft, levels 2-24} \end{cases}$ Bay 2: $M_{pm} = \begin{cases} 17.4 \text{ k-ft, roof} \\ 29.8 \text{ k-ft, levels 2-24} \end{cases}$

(c) For $G < 8$, $R = 1.43\left[1 + \frac{(2,6)}{8}\right]^2$; $G > 8$, $R = .714(2,6)$

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B GIRDER MOMENTS AT JOINTS	TABLE 9.14

	(1)	(2)		(3)	(4)	(5)	(6)		(7)	(8)
	Bays 1,3					Bay 2				
Level	Leeward End Moment, M_B Kip-ft.	Windward End Moment M_A Kip-ft	Maximum Positive Moment, M_C Kip-ft	Joint Moment, M_{jB} Kip-ft	Joint Moment, M_{jA} Kip-ft	Leeward End Moment, M_B Kip-ft.	Windward End Moment, M_A Kip-ft.		Joint Moment, M_{jB} Kip-ft	Joint Moment, M_{ja} Kip-ft
	Tab. 9.13(4)	Tab. 9.13(2) x M_{pm} - (1)	C x N_p	Note a	Tab. 9.13(1) - (3)	Tab. 9.13(8)	Tab 9.13(6) x M_{pm} - (5)		Note a	Tab. 9.13(1) - (7)
R	150	-132		168	-149	31.3	-14.4		39.7	-21.0
2	170	-105		190	-122	67.3	-6.2		83.5	-15.8
3	191	-72.8		212	-88.0	92.1	19.9		111	13.0
4	212	-43.9		234	-58.0	118	40.8		139	37.0
5	234	-14.9		257	-28.0	149	57.8		173	56.0
6	258	11.0		282	-1.0	181	72.6		208	73.0
7	282	38.0		308	26.0	215	86.0		244	90.0
8	308	60.9		335	51.0	249	100		281	105
9	335	85.8		363	77.0	283	113		317	123
10	363	108		392	101	316	128		353	140
11	392	130		422	123	352	140		391	154
12	422	150		453	145	384	155		426	172
13	452	170		484	166	420	167		465	185
14	485	187		518	184	453	182		500	202
15	519	203		554	201	489	193		539	216
16	552	220		588	219	519	208		571	236
17	587	235		624	235	554	221		609	250
18	623	249		661	251	587	235		644	268
19	658	264		697	267	620	250		680	284
20	696	278		736	282	656	262		718	300
21	735	294		777	295	691	278		756	316
22	771	309		814	310	724	292		792	332
23	910	363		957	369	855	343		933	393
24	1039	418		1090	436	983	394		1070	456

(a) $M_{jB} = (1,5) + \left[4 + \frac{\text{Tab. 9.13}(2,6)}{2}\right] M_{pm} \left(\frac{d_c/L}{1-d_c/L}\right)$

M_{pm} = Tab. 9.13, note (b) ← should have used 101.9

Bays 1,3: $\frac{d_c/L}{1-d_c/L} = \frac{14/(27\times12)}{1-14/(27\times12)} = .045$

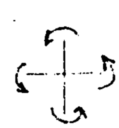
Bay 2: $\frac{d_c/L}{1-d_c/L} = \frac{14/(12\times12)}{1-14/(12\times12)} = .108$

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B MOMENT BALANCE OF JOINT 1	TABLE 9.15

	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Level	Initial Column ~~End~~ Joint Moments Kip-ft.		Unbalanced Moment at Joint Kip-ft	Change in Column ~~End~~ Joint Moment Kip-ft.		Column ~~End~~ Joint Moment Kip-ft	
	Above	Below		Above	Below	Above	Below
	Tab. 9.12(10) × DF_1 Note (a)	Tab. 9.12(11) × DF_2 Note (a)	−[Tab 9.14(4) + (1) + (2)] Note (b)	(3) × DF_3 Note (c)	(3) × DF_4 Note (c)	(1) + (4)	(2) + (5)
R	0	-14.0	163	0	163	0	149
2	-14.0	-36.8	173	86.5	86.5	72.5	49.7
3	-36.8	-56.0	181	90.5	90.5	53.7	34.5
4	-56.0	-76.0	190	95.0	95.0	39.0	19.0
5	-76.0	-95.8	200	100	100	24.0	4.2
6	-95.8	-116	213	107	107	11.2	-9.0
7	-116	-135	225	113	113	-3.0	-22.0
8	-135	-155	239	120	120	-15.0	-35.0
9	-155	-175	253	127	127	-28.0	-48.0
10	-175	-195	269	135	135	-40.0	-60.0
11	-195	-214	286	143	143	-52.0	-71.0
12	-214	-234	303	152	152	-62.0	-82.0
13	-234	-254	322	161	161	-73.0	-93.0
14	-254	-273	343	172	172	-82.0	-101
15	-273	-293	365	183	183	-90.0	-110
16	-293	-313	387	194	194	-99.0	-119
17	-313	-332	410	205	205	-108	-127
18	-332	-352	433	217	217	-115	-135
19	-352	-371	456	228	228	-124	-143
20	-371	-392	481	241	241	-130	-151
21	-392	-412	509	255	255	-137	-157
22	-412	-432	534	267	267	-145	-165
23	-432	-563	626	313	313	-119	-250
24	-563	-582	709	355	355	-208	-227
Foundation Moment	(8) Initial Column Moment = -582			(9) Carry-over Moment = +178		(10) Final Moment = -404	

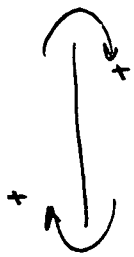
(a) Assume $DF_1 = DF_2 = 0.25$

(b) + Joint moment:

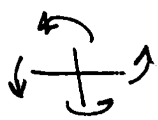


(c) Roof: $DF_3 = 0$, $DF_4 = 1.0$ Levels 2-24: $DF_3 = 0.5$, $DF_4 = 0.5$

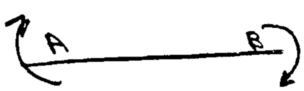
Sign Conventions



← + col. m's



← + joint m's



← + girder m's

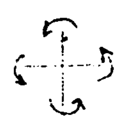
DESIGN EXAMPLE - PART 2
WIND RESISTANT BENT E
MOMENT BALANCE OF JOINT 2

TABLE 9.16

Level	(1) Initial Column Joint Moments Kip-ft. Above	(2) Initial Column Joint Moments Kip-ft. Below	(3) Unbalanced Moment at Joint Kip-ft	(4) Change in Column Joint Moment Kip-ft. Above	(5) Change in Column Joint Moment Kip-ft. Below	(6) Column Joint Moment Kip-ft Above	(7) Column Joint Moment Kip-ft Below
	Tab. 9.12(10) × DF_1 Note (a)	Tab. 9.12(11) × DF_2 Note (a)	−[Tab. 9.14(3+8) + (1) + (2)] Note (b)	(3) × DF_3 Note (c)	(2) × DF_4 Note (c)	(1) + (4)	(2) + (5)
R	0	−14.0	−133	0	−133	0	−147
2	−14.0	−36.8	−123	−61.5	−61.5	−75.5	−98.3
3	−36.8	−56.0	−132	−66.0	−66.0	−103	−122
4	−56.0	−76.0	−139	−69.5	−69.5	−126	−146
5	−76.0	−95.8	−141	−70.5	−70.5	−147	−166
6	−95.8	−116	−143	−71.5	−71.5	−167	−188
7	−116	−135	−147	−73.5	−73.5	−190	−209
8	−135	−155	−150	−75.0	−75.0	−210	−230
9	−155	−175	−156	−78.0	−78.0	−233	−253
10	−175	−195	−162	−81.0	−81.0	−256	−276
11	−195	−214	−167	−83.5	−83.5	−279	−298
12	−214	−234	−177	−88.5	−88.5	−303	−323
13	−234	−254	−181	−90.5	−90.5	−325	−345
14	−254	−273	−193	−96.5	−96.5	−351	−370
15	−273	−293	−204	−102	−102	−375	−395
16	−293	−313	−218	−109	−109	−402	−422
17	−313	−332	−229	−115	−115	−428	−447
18	−332	−352	−245	−123	−123	−455	−475
19	−352	−371	−258	−129	−129	−481	−500
20	−371	−392	−273	−137	−137	−508	−529
21	−392	−412	−289	−145	−145	−537	−557
22	−412	−432	−302	−151	−151	−563	−583
23	−432	−563	−355	−178	−178	−610	−741
24	−563	−582	−401	−201	−201	−764	−783
Foundation Moment	(8) Initial Column Moment = −582		(9) Carry-over Moment = −101			(10) Final Moment = −683	

(a) Assume $DF_1 = DF_2 = 0.2$[illegible]

(b) + Joint moments:



(c) Roof: $DF_3 = 0$, $DF_4 = 1.0$ Levels 2-24: $DF_3 = 0.5$, $DF_4 = 0.5$

DESIGN EXAMPLE - PART 2
WIND RESISTANT BENT B
MOMENT BALANCE OF JOINT 3

TABLE 9.17

4+7

Level	(1) Initial Column Joint ~~End~~ Moments Kip-ft. Above: Tab. 9.12(10) × DF_1 Note (a)	(2) Initial Column Joint ~~End~~ Moments Kip-ft. Below: Tab. 9.12(11) × DF_2 Note (a)	(3) Unbalanced Moment at Joint Kip-ft: −[Tab. 9.14 + (1) + (2)] Note (b)	(4) Change in Column Joint ~~End~~ Moment Kip-ft. Above: (3) × DF_3 Note (c)	(5) Change in Column Joint ~~End~~ Moment Kip-ft. Below: (3) × DF_4 Note (c)	(6) Column Joint ~~End~~ Moment Kip-ft Above: (1) + (4)	(7) Column Joint ~~End~~ Moment Kip-ft Below: (2) + (5)
R	0	-14.0	123	0	123	0	109
2	-14.0	-36.8	89.3	44.7	44.7	30.7	7.9
3	-36.8	-56.0	69.8	34.9	34.9	-1.9	-21.1
4	-56.0	-76.0	51.0	25.5	25.5	-30.5	-50.5
5	-76.0	-95.8	26.8	13.4	13.4	-62.6	-82.4
6	-95.8	-116	4.8	2.4	2.4	-93.4	-114
7	-116	-135	-19.0	-9.5	-9.5	-126	-145
8	-135	-155	-42.0	-21.0	-21.0	-156	-176
9	-155	-175	-64.0	-32.0	-32.0	-187	-207
10	-175	-195	-84.0	-42.0	-42.0	-217	-237
11	-195	-214	-105	-52.5	-52.5	-248	-267
12	-214	-234	-123	-61.5	-61.5	-276	-296
13	-234	-254	-143	-71.5	-71.5	-306	-326
14	-254	-273	-157	-78.5	-78.5	-333	-352
15	-273	-293	-174	-87.0	-87.0	-360	-380
16	-293	-313	-184	-92.0	-92.0	-385	-405
17	-313	-332	-199	-100	-100	-413	-432
18	-332	-352	-211	-106	-106	-438	-458
19	-352	-371	-224	-112	-112	-464	-483
20	-371	-392	-237	-119	-119	-490	-511
21	-392	-412	-247	-124	-124	-516	-536
22	-412	-432	-258	-129	-129	-541	-561
23	-432	-563	-307	-154	-154	-586	-717
24	-563	-582	-361	-181	-181	-744	-763

Foundation Moment: (8) Initial Column Moment = -582; (9) Carry-over Moment = -91; (10) Final Moment = -673

(a) Assume $DF_1 = DF_2 = 0.25$

(b) + Joint moments:

(c) Roof: $DF_3 = 0$, $DF_4 = 1.0$ Levels 2-24: $DF_3 = 0.5$, $DF_4 = 0.5$

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B MOMENT BALANCE OF JOINT 4	TABLE 9.18

	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Level	Initial Column Joint Moments Kip-ft. Above	Initial Column Joint Moments Kip-ft. Below	Unbalanced Moment at Joint Kip-ft	Change in Column Joint Moment Kip-ft. Above	Change in Column Joint Moment Kip-ft. Below	Column Joint Moment Kip-ft Above	Column Joint Moment Kip-ft Below
	Tab. 9.12(10) × DF_1 Note (a)	Tab. 9.12(11) × DF_2 Note (a)	−[Tab. 9.14(3) + (1) + (2)] Note (b)	(3) × DF_3 Note (c)	(2) × DF_4 Note (c)	(1) + (4)	(2) + (5)
R	0	−14.0	−154	0	−154	0	−168
2	−14.0	−36.8	−139	−69.5	−69.5	−83.5	−106
3	−36.8	−56.0	−119	−59.5	−59.5	−96.3	−116
4	−56.0	−76.0	−102	−51.0	−51.0	−107	−127
5	−76.0	−95.8	−85.2	−42.6	−42.6	−119	−138
6	−95.8	−116	−70.2	−35.1	−35.1	−131	−151
7	−116	−135	−57.0	−28.5	−28.5	−145	−164
8	−135	−155	−45.0	−22.5	−22.5	−158	−178
9	−155	−175	−33.0	−16.5	−16.5	−172	−192
10	−175	−195	−22.0	−11.0	−11.0	−186	−206
11	−195	−214	−13.0	−6.5	−6.5	−202	−221
12	−214	−234	−5.0	−2.5	−2.5	−217	−237
13	−234	−254	4.0	2.0	2.0	−232	−252
14	−254	−273	9.0	4.5	4.5	−250	−269
15	−273	−293	12.0	6.0	6.0	−267	−287
16	−293	−313	18.0	9.0	9.0	−284	−304
17	−313	−332	21.0	10.5	10.5	−303	−322
18	−332	−352	23.0	11.5	11.5	−321	−341
19	−352	−371	26.0	13.0	13.0	−339	−358
20	−371	−392	27.0	13.5	13.5	−358	−379
21	−392	−412	27.0	13.5	13.5	−379	−399
22	−412	−432	30.0	15.0	15.0	−397	−417
23	−432	−563	38.0	19.0	19.0	−413	−544
24	−563	−582	55.0	27.5	27.5	−536	−555
Foundation Moment	(8) Initial Column Moment = −582		(9) Carry-over Moment = 14		(10) Final Moment = −568		

(a) Assume $DF_1 = DF_2 = 0.25$

(b) + Joint moment:

(c) Roof: $DF_3 = 0$, $DF_4 = 1.0$ Levels 2-24: $DF_3 = 0.5$, $DF_4 = 0.5$

DESIGN EXAMPLE – PART 2
WIND RESISTANT BENT B
EXTERIOR COL. THRUSTS, COMBINED LOADING (F=1.3)

TABLE 9.19

Level	(1) Σ Girder Joint Moments Each Bay (F=1.3). Kip-ft.	(2) Vertical Girder Shears, Bays 1 and 3 (F=1.3) Kips	(3) Σ Vertical Girder Shears (F=1.3) Kips	(4) Column Thrust, Gravity Loads (F=1.3) Kips	(5) Left Col. Thrust, Combined Loading (F=1.3) Kips	(6) Right Col. Thrust, Combined Loading (F=1.3) Kips
	Tab. 9.13(1)	(1)/L	Σ(2)	Tab. 9.11(5)	(4)−(3)	(4)+(3)
R	18.7	0.7	0.7	43	42	44
2	67.7	2.5	3.2	103	100	106
3	124	4.6	7.8	153	145	161
4	176	6.5	14.3	216	202	230
5	229	8.5	22.8	269	246	292
6	281	10.4	33.2	322	289	355
7	334	12.4	45.6	374	328	420
8	386	14.3	59.9	428	368	488
9	440	16.3	76.2	481	405	557
10	493	18.3	94.5	534	439	629
11	545	20.2	115	588	473	703
12	598	22.1	137	640	503	777
13	650	24.1	161	693	532	854
14	702	26.0	187	746	559	933
15	755	28.0	215	800	585	1015
16	807	29.9	245	853	608	1098
17	859	31.8	277	905	628	1182
18	912	33.8	311	958	647	1269
19	964	35.7	347	1011	664	1358
20	1018	37.7	385	1065	680	1450
21	1072	39.7	425	1118	693	1543
22	1124	41.6	467	1170	703	1637
23	1326	49.1	516	1225	709	1741
24	1526	56.5	573	1283	710	1856

DESIGN EXAMPLE - PART 2
WIND RESISTANT BENT B
INTERIOR COL. THRUSTS, COMBINED LOADING (F=1.3)

TABLE 9.20

Level	(1) Σ Girder Joint Moments Each Bay (F=1.3) Kip-ft.	(2) VERTICAL Girder Shears, Bay 2 (F=1.3) Kips	(3) Σ Vertical Girder Shears (F=1.3) Kips	(4) Column Thrust, Gravity Loads (F=1.3) Kips	(5) Left Col. Thrust, Combined Loading (F=1.3) Kips	(6) Right Col. Thrust, Combined Loading (F=1.3) Kips
	Tab. 9.13(1)	(1)/L	Σ(2)	Tab. 9.11(10)	(4) - (3) + Tab. 9.19(3)	(4) + (3) - Tab 9.19(3)
R	18.7	1.6	1.6	49	48	50
2	67.7	5.6	7.2	111	107	115
3	124	10.3	17.5	157	147	167
4	176	14.7	32.2	213	195	231
5	229	19.1	51.3	267	239	295
6	281	23.4	74.7	321	280	362
7	334	27.8	103	376	319	433
8	386	32.2	135	430	355	505
9	440	36.7	171	485	390	580
10	493	41.1	213	540	422	658
11	545	45.4	258	594	451	737
12	598	49.8	308	649	478	820
13	650	54.2	362	703	502	904
14	702	58.5	420	758	525	991
15	755	62.9	483	813	545	1081
16	807	67.3	551	867	561	1173
17	859	71.6	622	922	577	1267
18	912	76.0	698	976	589	1363
19	964	80.3	779	1031	599	1463
20	1018	84.8	863	1086	608	1564
21	1072	89.3	953	1140	612	1668
22	1124	93.7	1046	1195	616	1774
23	1326	111	1157	1249	608	1890
24	1526	127	1284	1304	593	2015

DESIGN EXAMPLE-PART 2 WIND RESISTANT BENT B GRAVITY MOMENTS (F=1.7)	TABLE 9.21

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	BAYS 1, 3				BAY 2			
Level	Girder End Moments, M_B & $-M_A$ Kip-ft	Σ Girder Joint Moments, Joint 1 Kip-ft	Column 1 Joint Moments Above Kip-ft	Column 1 Joint Moments Below Kip-ft	Girder End Moments, M_B, $-M_A$ Kip-ft	Σ Girder Joint Moments, Joint 2 Kip-ft	Column 2 Joint Moments Above Kip-ft	Column 2 Joint Moments Below Kip-ft
	$\frac{1.7 w L_g^2}{12}$	$-(1)-\frac{1.7}{1.3}\times 4\times M_{pm}\times\frac{d_c/L}{1-d_c/L}$ (a)	$-(2)\times DF_1$ (b)	$-(2)\times DF_2$ (b)	$\frac{1.7 w L_g^2}{12}$	$-(2)-(5)-\frac{1.7}{1.3}\times 4\times M_{pm}\times\frac{d_c/L}{1-d_c/L}$	$-(6)\times DF_1$	$-(6)\times DF_2$
R	174	-198	0	198	30.3	158	0	-158
2	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
3	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
4	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
5	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
6	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
7	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
8	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
9	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
10	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
11	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
12	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
13	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
14	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
15	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
16	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
17	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
18	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
19	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
20	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
21	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
22	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
23	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
24	172	-195	97.5	97.5	52.0	126	-63.0	-63.0
	Foundation Carry-over Moment = 48.8				Foundation Carry-Over Moment = -31.5			

(a) Bays 1,3: $M_{pm} = \begin{cases} 99.8 \text{ k-ft., roof} \\ 98.7 \text{ k-ft., levels 2-24} \end{cases}$ Bay 2: $M_{pm} = \begin{cases} 17.4 \text{ k-ft., roof} \\ 29.8 \text{ k-ft, levels 2-24} \end{cases}$

(b) Roof: $DF_1 = 0$, $DF_2 = 1.0$ Levels 2-24: $DF_1 = DF_2 = 0.5$

DESIGN EXAMPLE – PART 2
WIND RESISTANT BENT B
TRIAL EXTERIOR COLUMN SELECTION, COMBINED LOADING (F=1.3)

TABLE 9.22

Joint	Level	(1) Column Joint Moment Kip-ft. Above	(2) Column Joint Moment Kip-ft. Below	(3) Girder Depth ft.	(4) Column End Moment Kip-ft. Above	(5) Column End Moment Kip-ft. Below	(6) Column Moments Below Level Kip-ft. Top	(7) Column Moments Below Level Kip-ft. Bottom	(8) Axial Load Kips	(9) Trial P_Y Kips (a)	(10) Trial Section W-Shapes A36	(11) Trial Section W-Shapes A572
		Tabs. 9.15 & 9.18	Tabs. 9.15 & 9.18	Fig. 9.	$(1)-\frac{(3)}{2}\times\left[\frac{(1)+(2)}{h}\right]$	$(2)-\frac{(3)}{2}\times\left[\frac{(1)+(2)}{h}\right]$	(5), Level n	(4), Level n+1	Tab. 9.19	$(8)+2.1\times\frac{(6\text{ or }7)}{d}$ OR $1.33\times(8)$	DA I	DA I
1	R	0	149	1.33	-10.3	139	139	63.0	42.0	291		
	2	72.5	49.7	1.50	63.0	40.2	40.2	46.9	100	184		
	3	53.7	34.5	1.50	46.9	27.7						
	4	39.0	19.0	1.50	34.5	14.5	14.5	21.8	202	269*		
	5	24.0	4.2	1.50	21.8	2.0						
	8	-15.0	-35.0	1.75	-9.8	-29.8	-29.8	-20.1	368	489*		
	9	-28.0	-48.0	2.0	-20.1	-40.1						
	12	-62.0	-82.0	2.0	-47.1	-67.1	-67.1	-55.8	503	669*		
	13	-73.0	-93.0	2.0	-55.8	-75.8						
	16	-99.0	-119	2.0	-76.5	-96.5	-96.5	-83.7	608	809*		
	17	-108	-127	2.0	-83.7	-103						
	20	-130	-151	2.25	-97.3	-118	-118	-103	680	904*		
	21	-137	-157	2.25	-103	-123						
	24	-208	-227	2.50	-163	-182	-182	-404	710	1435		
	F				-404							
4	R	0	-168	1.33	11.6	-156	-156	-68.8	44.0	324		
	2	-83.5	-106	1.50	-68.8	-91.3	-91.3	-79.8	106	270		
	3	-96.3	-116	1.50	-79.8	-99.5						
	4	-107	-127	1.50	-88.9	-109	-109	-99.1	230	426		
	5	-119	-138	1.50	-99.1	-118						
	8	-158	-178	1.75	-128	-148	-148	-134	488	754	12x79	
	9	-172	-192	2.0	-134	-154						
	12	-217	-237	2.0	-170	-190	-190	-182	777	1118	12x106	
	13	-232	-252	2.0	-182	-202						
	16	-284	-304	2.0	-223	-243	-243	-238	1098	1534	14x150	12x106
	17	-303	-322	2.0	-238	-257						
	20	-358	-379	2.25	-272	-293	-293	-288	1450	1976	14x193	14x136
	21	-379	-399	2.25	-288	-308						
	24	-536	-555	2.50	-422	-441	-441	-568	1856	2875	14x287	14x202
	F				-568							

(a) When the value of the trial P_Y is governed by the condition that $P/P_Y \le 0.75$, an * will indicate as such.

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B TRIAL INTERIOR COLUMN SELECTION, COMBINED LOADING (F=1.3)	TABLE 9.23

		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Joint	Level	Column Joint Moment Kip-ft.		Girder Depth ft.	Column End Moment Kip-ft.		Column Moments Below Level Kip-ft.		Axial Load Kips	Trial P_Y Kips	Trial Section W-Shapes	
		Above	Below		Above	Below	Top	Bottom		(a)	A36	A572
		Tabs. 9.16 & 9.17	Tabs. 9.16 & 9.17	Fig 9.	$(1)-\frac{(3)}{2}\times\left[\frac{(1)+(2)}{h}\right]$	$(2)-\frac{(3)}{2}\times\left[\frac{(1)+(2)}{h}\right]$	(5), Level n	(4), Level n+1	Tab. 9.20	$(8)+2.1\times\frac{(6\text{ or }7)}{d}$ OR $1.33\times(8)$	DA I	DA I
	R	0	-147	0.83	6.3	-141	-141	-66.5	48	301		
	2	-75.5	-98.3	1.0	-66.5	-89.3	-89.3	-89.4	107	267	12x40	
	3	-103	-122	1.17	-89.4	-108						
	4	-126	-146	1.17	-110	-130	-130	-125	195	428	12x40	
	5	-147	-166	1.33	-125	-144						
	8	-210	-230	1.75	-170	-190	-190	-189	355	695		
	9	-233	-253	1.75	-189	-209						
2	12	-303	-323	2.0	-238	-258	-258	-256	478	940		
	13	-325	-345	2.0	-256	-276						
	16	-402	-422	2.0	-317	-337	-337	-338	561	1166		
	17	-428	-447	2.0	-338	-357						
	20	-508	-529	2.0	-400	-421	-421	-410	608	1362		
	21	-537	-557	2.25	-410	-430						
	24	-764	-783	2.50	-603	-622	-622	-683	593	1816		
	F				-683							
	R	0	109	0.83	-4.7	104	104	28.7	50	236		
	2	30.7	7.9	1.0	28.7	5.9	5.9	-0.5	115	153*		
	3	-1.9	-21.1	1.17	-0.5	-19.7						
	4	-30.5	-50.5	1.17	-25.6	-45.6	-45.6	-52.6	231	325		
	5	-62.6	-82.4	1.33	-52.6	-72.4						
	8	-156	-176	1.75	-126	-146	-146	-151	505	775	12x79	
	9	-187	-207	1.75	-151	-171						
3	12	-276	-296	2.0	-217	-237	-237	-241	820	1251	12x120	
	13	-306	-326	2.0	-241	-261						
	16	-385	-405	2.0	-303	-323	-323	-326	1173	1757	14x167	12x120
	17	-413	-432	2.0	-326	-345						
	20	-490	-511	2.0	-386	-407	-407	-394	1564	2293	14x219	14x158
	21	-516	-536	2.25	-394	-414						
	24	-744	-763	2.50	-587	-606	-606	-673	2015	3220	14x314	14x219
	F				-673							

(a) When the value of the trial P_Y is governed by the condition that $P/P_Y \leq 0.75$, an * will indicate as such.

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B TRIAL COLUMN SELECTION, GRAVITY LOADING (F=1.7)	TABLE 9.24

Joint	Level	(1) Column Joint Moment Kip-ft. Above	(2) Column Joint Moment Kip-ft. Below	(3) Girder Depth ft.	(4) Column End Moment Kip-ft. Above	(5) Column End Moment Kip-ft. Below	(6) Column Moments Below Level Kip-ft. Top	(7) Column Moments Below Level Kip-ft. Bottom	(8) Axial Load Kips	(9) Trial P_y Kips (a)	(10) Trial Section W-Shapes A36	(11) Trial Section W-Shapes A572
		Tab. 9.21	Tab. 9.21	Fig. 9.	$(1)-\frac{(3)}{2}\times\left[\frac{(1)+(2)}{h}\right]$	$(2)-\frac{(3)}{2}\times\left[\frac{(1)+(2)}{h}\right]$	(5), Level n	(4), Level n+1	Tab. 9.11	$(8)+2.1\times\frac{(6\text{ or }7)}{d}$ OR $1.33\times(8)$	DA I	DA I
1 & 4	R	0	198	1.33	-13.6	184	184	82.4	56	386	12x45	
	2	97.5	97.5	1.50	82.4	82.4	82.4	82.4	134	282	12x45	
	3	97.5	97.5	1.50	82.4	82.4						
	4	97.5	97.5	1.50	82.4	82.4	82.4	82.4	282	430	12x45	
	5	97.5	97.5	1.50	82.4	82.4						
	8	97.5	97.5	1.75	79.9	79.9	79.9	77.3	559	743*		
	9	97.5	97.5	2.0	77.3	77.3						
	12	97.5	97.5	2.0	77.3	77.3	77.3	77.3	836	1112*		
	13	97.5	97.5	2.0	77.3	77.3						
	16	97.5	97.5	2.0	77.3	77.3	77.3	77.3	1115	1483*		
	17	97.5	97.5	2.0	77.3	77.3						
	20	97.5	97.5	2.25	74.8	74.8	74.8	74.8	1392	1851*		
	21	97.5	97.5	2.25	74.8	74.8						
	24	97.5	97.5	2.50	76.9	76.9	76.9	48.8	1678	2232*		
	F				48.8							
2 & 3	R	0	-158	0.83	6.8	-151	-151	-56.5	65	336	12x40	
	2	-63.0	-63.0	1.0	-56.5	-56.5	-56.5	-56.5	145	246		
	3	-63.0	-63.0	1.17	-55.4	-55.4						
	4	-63.0	-63.0	1.17	-55.4	-55.4	-55.4	-54.3	279	378		
	5	-63.0	-63.0	1.33	-54.3	-54.3						
	8	-63.0	-63.0	1.75	-51.6	-51.6	-51.6	-51.6	563	749*		
	9	-63.0	-63.0	1.75	-51.6	-51.6						
	12	-63.0	-63.0	2.0	-50.0	-50.0	-50.0	-50.0	848	1128*		
	13	-63.0	-63.0	2.0	-50.0	-50.0						
	16	-63.0	-63.0	2.0	-50.0	-50.0	-50.0	-50.0	1134	1508*		
	17	-63.0	-63.0	2.0	-50.0	-50.0						
	20	-63.0	-63.0	2.0	-50.0	-50.0	-50.0	-48.4	1420	1889*		
	21	-63.0	-63.0	2.25	-48.4	-48.4						
	24	-63.0	-63.0	2.50	-49.9	-49.9	-49.9	-31.5	1705	2268*		
	F				-31.5							

(a) When the value of the trial P_y is governed by the condition that $P/P_y \leq 0.75$, an * will indicate as such.

DESIGN EXAMPLE - PART 2
WIND RESISTANT BENT B
EXTERIOR COLUMNS, - FINAL DESIGN, A36 STEEL

TABLE 9.25

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Below Level	Req'd P Kips Req'd M Kip-ft.	Trial Section W-Shape	h ft. Ratio g	P_Y Kips M_P Kip-ft.	r_x in. r_y in.	h/r_x h/r_y	P/P_Y M_{pc}/M_p	Allow. M/M_{pc} Allow M Kip-ft.	Remarks
	Tabs. 9.22, 24 Tabs. 9.22, 24	DA-I	Fig. 9. Tabs. 9.22, 24	DA-I DA-I	DA-I DA-I	$\frac{12\times(3)}{(6)}$ $\frac{12\times(3)}{(6)}$	(1)/(4) $1.18\times(1-P/P_Y)$	DA-III (8)x(7)x(4)	
R	56 184	12x45	9.67 +.45	477 195	5.15 1.94	23 60	.12 1.0	1.0 195	>184 o.K.
2	134 82.4	12x45	9.67 +1.0	477 195	5.15 1.94	23 60	.28 .85	1.0 166	>82.4 o.K.
4	282 82.4	12x45	9.67 +1.0	477 195	5.15 1.94	23 60	.59 .48	1.0 93.6	>82.4 o.K.
8	488 148	12x79	9.67 +.91	836 358	5.34 3.05	22 38	.58 .49	1.0 175	>148 o.K.
		~~12x58~~		614 260	5.28 2.51	22 46	.79		>.75 N.G.
12	777 190	~~12x106~~	9.67 +.96	1123 490	5.46 3.11	21 37	.69 .36	1.0 176	<190 N.G.
		12x120		1271 559	5.51 3.13	21 37	.61 .46	1.0 257	>190 o.K.
16	1098 243	~~14x150~~	9.67 +.98	1587 811	6.37 3.99	18 29	.69 .36	1.0 292	>243 o.K.
		14x142	-	1507 765	6.32 3.97	18 29	.73 .32	1.0 245	>243 o.K.
20	1450 293	~~14x193~~	9.67 +.98	2042 1065	6.51 4.05	18 29	.71 .34	1.0 364	>293 o.K.
		14x184		1947 1013	6.49 4.04	18 29	.74 .30	1.0 305	>293 o.K.
24	1856 568	~~14x287~~	12.0 +.78	3037 1655	6.81 4.17	21 35	.61 .46	1.0 761	>568 o.K.
		14x264		2795 1507	6.74 4.14	21 35	.66 .40	1.0 597	>568 o.K.

DESIGN EXAMPLE - PART 2
WIND RESISTANT BENT 8
INTERIOR COLUMNS - FINAL DESIGN, A36 STEEL

TABLE 9.26

Below Level	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	Req'd P Kips Req'd M Kip-ft.	Trial Section W-Shape	h ft. Ratio q	P_y kips M_p kip-ft.	r_x in. r_y in.	h/r_x h/r_y	P/P_y M_{pc}/M_p	Allow. M/M_{pc} Allow M kip-ft.	Remarks
	Tabs. 9.23, 24 Tabs. 9.23, 24	DA-I	Fig. 9. Tabs. 9.23, 24	DA-I DA-I	DA-I DA-I	$\frac{12\times(3)}{(6)}$ $\frac{12\times(3)}{(6)}$	(1)/(4) $1.18\times(1-P/P_y)$	DA-III (8)x(7)x(4)	
R	65 151	12x40	9.67 +.37	424 173	5.13 1.94	23 60	.15 1.0	1.0 173	>151 O.K.
2	107 89	12x40	9.67 +1.0	424 173	5.13 1.94	23 60	.25 .88	1.0 153	>89 O.K.
4	195 130	~~12x40~~	9.67 +.96	424 173	5.13 1.94	23 60	.46 .64	1.0 111	<130 N.G.
		12x45		477 195	5.15 1.94	23 60	.41 .70	1.0 136	>130 O.K.
8	505 151	12x79	9.67 +.97	836 358	5.34 3.05	22 38	.60 .47	1.0 167	>151 O.K.
		~~12x58~~		614 260	5.28 2.51	22 46	.82		>.75 N.G.
12	820 241	~~12x120~~	9.67 +.98	1271 559	5.51 3.13	21 37	.65 .42	1.0 234	<241 N.G.
		12x133		1408 629	5.59 3.16	21 37	.58 .49	1.0 310	>241 O.K.
16	1173 326	14x167	9.67 +.99	1767 909	6.42 4.01	18 29	.66 .40	1.0 361	>326 O.K.
		~~14x158~~		1673 859	6.40 4.00	18 29	.70 .35	1.0 303	<326 N.G.
20	1564 407	~~14x219~~	9.67 +.97	2317 1224	6.59 4.08	18 28	.68 .38	1.0 469	>407 O.K.
		14x211		2235 1175	6.56 4.07	18 29	.70 .35	1.0 416	>407 O.K.
24	2015 673	14x314	12.0 +.90	3323 1835	6.90 4.20	21 34	.61 .46	1.0 852	>673 O.K.
		~~14x287~~		3037 1655	6.81 4.17	21 35	.66 .40	1.0 657	<673 N.G.

DESIGN EXAMPLE - PART 2
WIND RESISTANT BENT B
EXTERIOR & INTERIOR COLUMNS - FINAL DESIGN, A572 STEEL

TABLE 9.27

Below Level	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	Req'd P Kips Req'd M Kip-ft.	Trial Section W-Shape	h ft. Ratio β	P_Y Kips M_P Kip-ft.	r_x in. r_y in.	h/r_x h/r_y	P/P_Y M_{pc}/M_p	Allow. M/M_{pc} Allow M Kip-ft.	Remarks
	Tabs. 9.22,23 Tabs. 9.22,23	DA-I	Fig. 9. Tabs. 9.22,23	DA-I DA-I	DA-I DA-I	$\frac{12\times(3)}{(6)}\times1.13$ $\frac{12\times(3)}{(6)}\times1.18$	(1)/(4) $1.18\times(1-P/P_Y)$	DA-III (8)x(7)x(4)	
		EXTERIOR COLUMNS							
16	1098 243	~~12x106~~	9.67 +.98	1560 681	5.46 3.11	25 44	.70 .35	1.0 238	< 243 N.G.
		12x120	--	1766 777	5.51 3.13	25 44	.62 .45	1.0 347	> 243 O.K.
20	1450 293	14x136	9.67 +.98	1999 1011	6.31 3.77	22 36	.73 .32	1.0 328	> 293 O.K.
		~~14x127~~		1867 941	6.29 3.76	22 36	.78		> .75 N.G.
24	1856 568	~~14x202~~	12.0 +.78	2970 1556	6.54 4.06	26 42	.62 .44	1.0 689	> 568 O.K.
		14x193		2836 1480	6.51 4.05	26 42	.65 .41	1.0 603	> 568 O.K.
		INTERIOR COLUMNS							
16	1173 326	~~12x120~~	9.67 +.99	1766 777	5.51 3.13	25 44	.66 .40	1.0 308	< 326 N.G.
		12x133		1956 874	5.59 3.16	25 43	.60 .47	1.0 413	> 326 O.K.
20	1564 407	14x158	9.67 +.97	2324 1193	6.40 4.00	21 34	.67 .39	1.0 460	> 407 O.K.
		~~14x150~~		2204 1125	6.37 3.99	22 34	.71 .34	1.0 385	< 407 N.G.
24	2015 673	~~14x219~~	12.0 +.90	3218 1700	6.59 4.08	26 42	.63 .44	1.0 750	> 673 O.K.
		14x211		3104 1632	6.56 4.07	26 42	.65 .41	1.0 676	> 673 O.K.

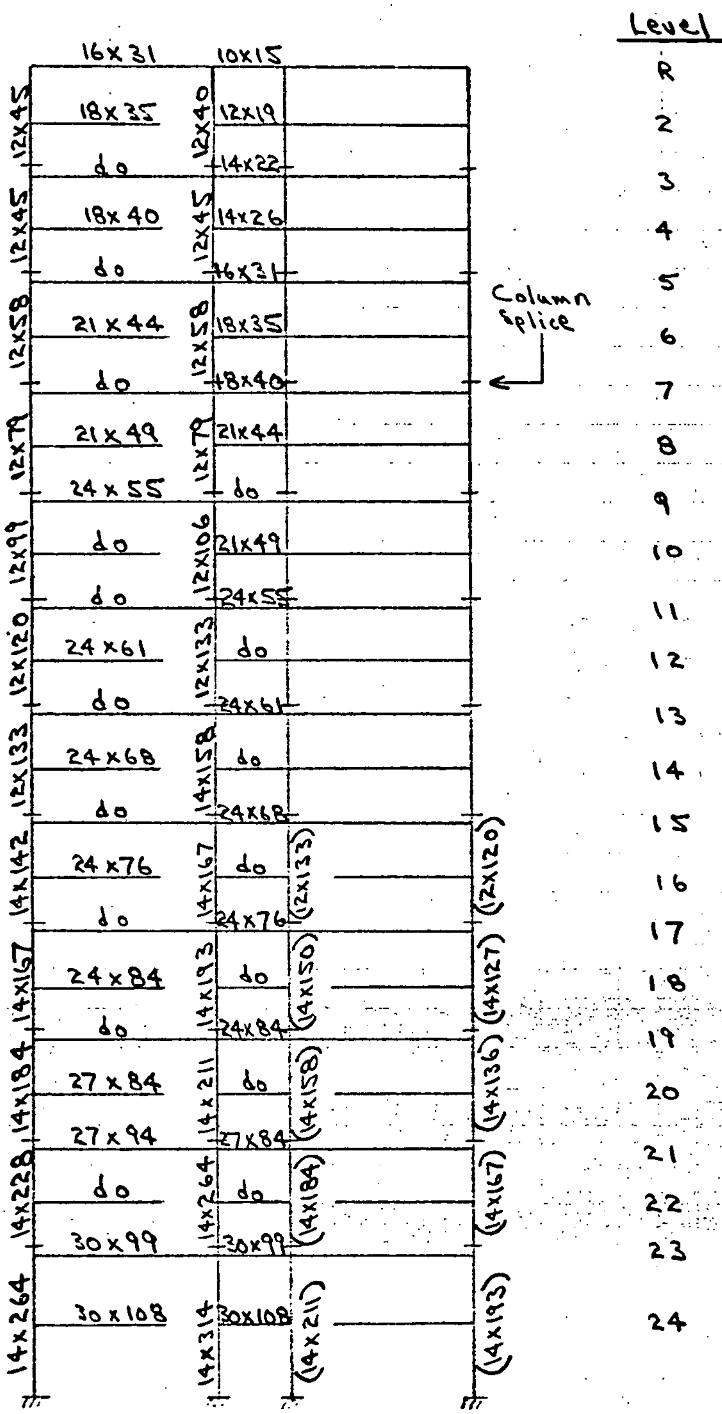
CORRECTIONS

1) Use small h for story ht.

2) Tab. 9.21, level R & Z → elastic gravity end moments exceed M_p → ∴ assume plastic hinges form at ends.

3) Put decimal pts. in the tables where a constant is used to distinguish from col. #'s in parens. L/2. or 2.0 etc.

DESIGN EXAMPLE - PART 2 WIND RESISTANT BENT B PRELIMINARY MEMBER SIZES	FIGURE 9.3



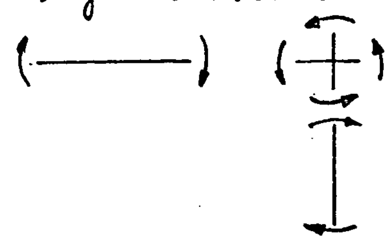
Notes:

(1) All sections are W-shapes

(2) All steel is A-36, except columns shown as (12x120), which are A572 steel, $F_y = 50$ ksi.

DESIGN EXAMPLE - PART 3 SUPPORTED BENT - A COLUMN CHECK - checkerboard Load	TABLE 9.28

sign convention +



Line	Item (1)	Units (2)	Operation (3)	Ext. Col. Level U,L Ext. Dead (4)	Ext. Col. Level C Int. Full (5)	Int. Col. Level U,L Int. Full (6)	Int. Col. Level U,L Ext. Dead (7)	Int. Col. Level C Int. Dead (8)	Int. Col. Level C Ext. Full (9)
1	Fw	K/f	Tab. 9.3 (14)		3.09	5.19			3.09
2	Fw_d	K/f	Tab. 9.3 (11,12) F=1.7	2.31			2.31	3.30	
3	L_g	ft	Tab. 9.2 (5)	26.0	11.0	11.0	26.0	11.0	26.0
4	dc	ft	Tab. 9.2 (4)	1.0	1.0	1.0	1.0	1.0	1.0
5	M_p or M_e	K-ft	Moment Diagram		0	-39.2	179.1	-33.3	179.1
6	M_d	K-ft	Eq. 7.2	0			179.1	-33.3	
7	M_j Full	K-ft	Eq. 7.1; (5)+$\frac{1}{4}$(1)(3)(4) OR 9.6(18)		4.5*	-53.5			202.7
8	M_j Dead	K-ft	Eq. 7.1; (6)+$\frac{1}{4}$(2)(3)(4) OR 9.6	-0.6*			197.6	-42.4	
9	Net Gird. Mom	K-ft	Eq. 7.1; (7)+(8)	-0.6	4.5	+144.1		+160.3	
10	Col. Mom.	K-ft	-0.5 × (9)	+0.3	-2.3	-72.1		-80.2	
11	q	—	(10) Level U ÷ (10) Level C	-0.13		+0.9 → +1.0			

* From 9.6(18)

Interior columns   q = 1.0

Checkerboad loading is the same as full gravity loading.

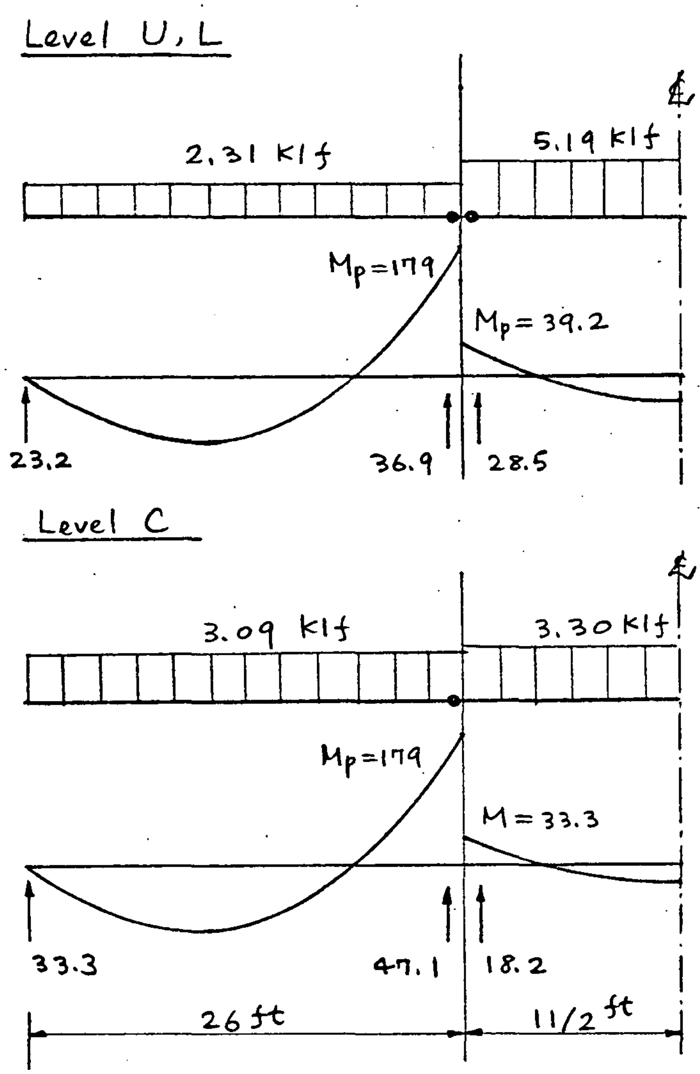
All columns OK. See Tab. 9.8(10).

DESIGN EXAMPLE - PART 3 SUPPORTED BENT - A COLUMN CHECK - Checkerboard Load	Table 9.28 cont'd

Exterior columns $q = -1$

From Tab. 9.7 (9), All $P/P_y \leq 0.90$; Tab. 9.7 (8), max. $h/r_x = 35$) $M/M_{pc} = 0.47$ by D.A. II-12

Maximum moment, 2.3 k-ft (Tab. 9. (10)), is less than the allowable moment ($M_p \times 0.47$).



Moment Diagram for Checkerboard Load

DESIGN EXAMPLE - PART 3 WIND RESISTANT BENT - B COLUMN CHECK, checkerboard Load	TABLE 9.29

Line	Item	Units	Operation	Level U, L		Level C	
				Int. Full	Ext. Dead	Int. Dead	Ext. Full
1	FW	K/f	Tab. 9.9 (5), F=1.7	5.32			3.20
2	FWd	K/f	Tab. 9.9 (2, 3), F=1.7		2.41	3.44	
3	M_j full	K-ft	Tab. 9.21 (2, 6)	-126			+195
4	M_j dead	K-ft	$(3) \times \frac{(2)}{(1)}$		+147	-81.3	
5	Net Gird. Mom.	K-ft	Eq. 7. , (3)+(4)	+21		+114	
6	Col. Mom.	K-ft	- 0.5 × (5)	-10.5		-57	
7	q	—	(10) Level U ÷ (10) Level C	+0.18 → 0			

Interior columns $q = 0$

From Tab. 9.26(7) all $P/P_y < 0.90$
Tab. 9.26(6) all $h/r_x < 25$) Bending strength same as full loading, Tab. 9.26(9)

From Tab. 9.26, all P/P_y and h/r_y fall below the curve in Fig. 7.
∴ LTB ok.

All allowable Moment, Tab. 9.26(8) > 57 K-ft, Tab. 9. (6)

All interior columns OK for checkerboard load.

DESIGN EXAMPLE - PART 3 WIND RESISTANT BENT - B COLUMN CHECK, Checkerboard Load	Table 9.29 Cont'd

Exterior columns $\quad q = 0$ assumed

From Tab. 9.25(7) All $P/P_y \leq .90$
Tab. 9.25(6) All $h/r_x \leq 25$) Bending Strength OK

From Tab. 9.25, All P/P_y and h/r_y fall below the curve in Fig. 7. , except a column below level 4.

Column bevel 4 ; $P/P_y = .59$, $h/r_y = 60$, then $M/M_{pc} = .825$ by D.A. III-2a.

From 9.24(5), Req'd $M = 82.4^{k\text{-}ft}$
From 9.25(7), $M_{pc} = 195 \times 0.48 = 93.6^{k\text{-}ft}$, then

$82.4 > 93.6 \times 0.825 = 77.1^{k\text{-}ft}$ N.G. for LTB

Increase Size W12×45 → W12×58 ($M_p = 260^{k\text{-}ft}$, $Z_x = 86.5\ in^3$)

$M_{pc} = 260 \times 0.48 = 126^{k\text{-}ft}$ (Tab. 9.25(7))

$82.4 < 126 \times 0.825 = 104^{k\text{-}ft}$ ok for LTB

DESIGN EXAMPLE - PART 3 SUPPORTED AND WIND RESISTANT BENT - A, B GIRDER DEFLECTION, Working Live Load	TABLE 9.30

Satisfy Eq. $\delta / L_g = \frac{5}{384} \frac{W_L L_g^3}{EI} \leq \frac{1}{360}$

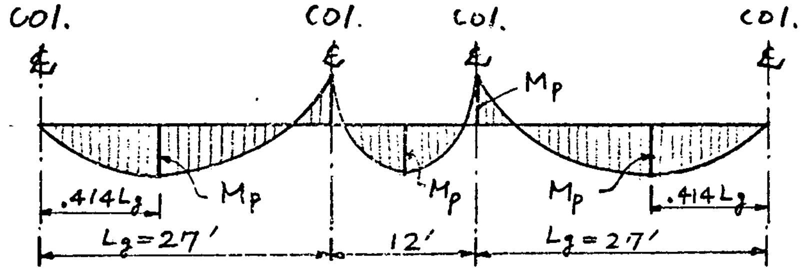
For $E = 29{,}000$ ksi, $\delta / L_g = 6.47 \times 10^{-5} \frac{W_L L_g^3}{I}$, where $I = in^4$, $W_L = ft$, $L_g = ft$

(1) SECTION	(2) Location	(3) Red. W_L k/f Tab. 9.1 Tab. 9.2 (2) Tab. 9.3 (14)	(4) L_g ft Tab. 9.2 (5)	(5) I in^4 Handbook	(6) L_g^3 ft^3 $(4)^3$	(7) $\delta/L_g \times 10^5$ $6.47 \times \frac{(3)(6)}{(5)}$	(8) Remarks
Supported Bent A							
W14 x 38	Roof / Ext.	.72	26.0	386	17 580	212	< 278
W 8 x 13	Roof / Int.	.72	11.0	39.6	1331	159	< 278
W 10 x 15	Floor / Int.	1.11	11.0	68.9	1331	139	< 278
Wind Resistant Bent B							
W16 x 31	Roof / Ext.	.72	26.0	374	17580	218	< 278
W10 x 15	Roof / Int.	.72	11.0	68.9	1331	90	< 278
W18 x 35	Floor / Ext.	.46	26.0	513	17580	101	< 278 [1)]
W12 x 19	Floor / Int.	1.11	11.0	130	1331	73	< 278 [1)]

Notes : (1) Lightest floor girder ∴ other members will be OK

DESIGN EXAMPLE - PART 3 SUPPORTED BENT A GIRDER LATERAL BRACING	TABLE 9.31

Moment Diagrams - drawn on tension side - at mechanism load

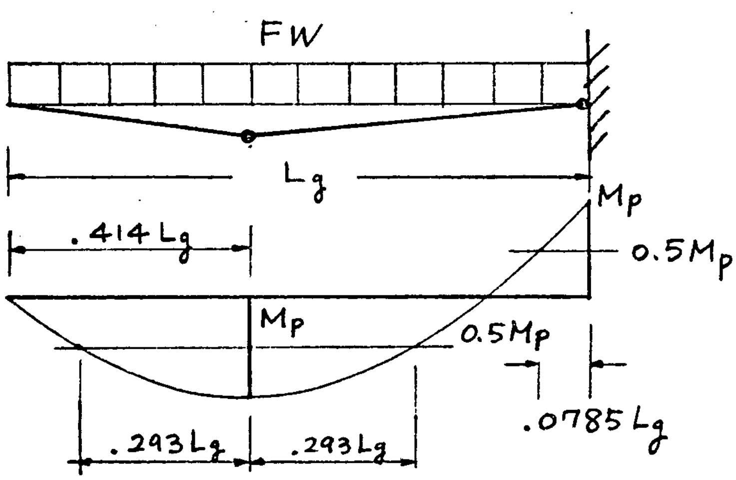


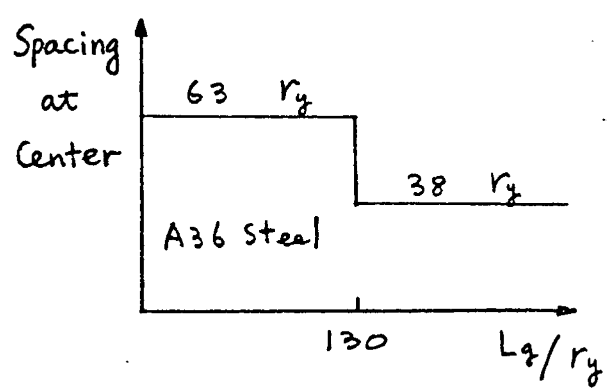
Tentative floor joist spacing : Ext. Bay - 3 ft. o.c.
Int. Bay - 2 ft. o.c.

<u>Joists are attached to the top flange of girders</u>

	(1)	(2)	(3)	(4)	(5)	(6)	(7)
LEVEL	SECTION	L_g ft	r_y in	$\frac{L_g \times 12'}{r_y}$	Max. Bracing Spacing Center	Ends	Remarks
R	W 8 × 13	11.0	.842	157	$38\ r_y = 32$	$63\ r_y = 53$	OK
2	W 10 × 15	11.0	.809	163	$38\ r_y = 30$	$63\ r_y = 51$	OK
R	W 14 × 38	26.0	1.54	202	$38\ r_y = 58$	$63\ r_y = 97$	OK

No additional bracing required.



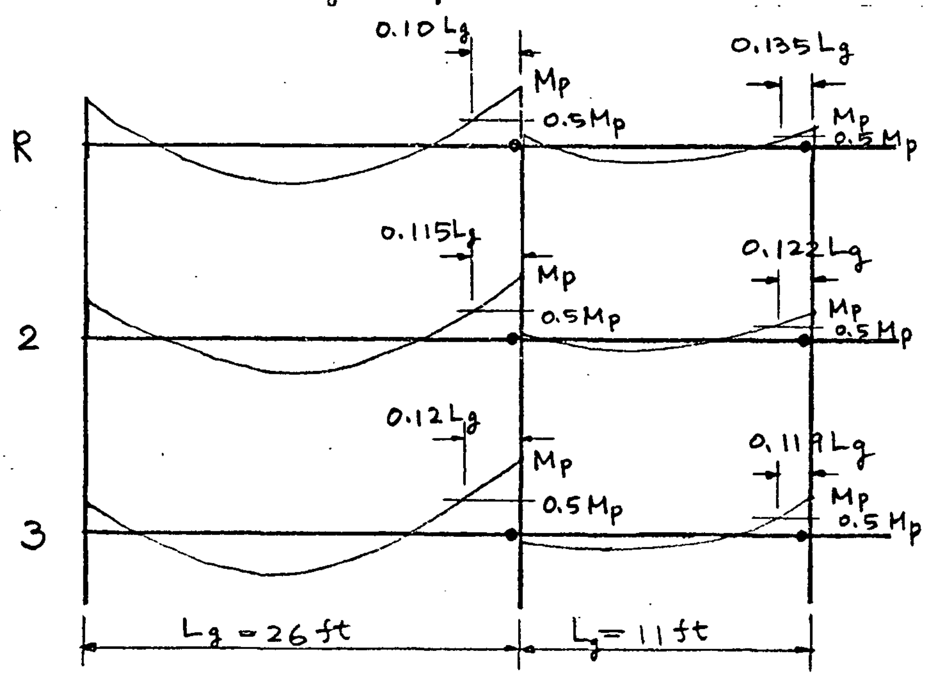


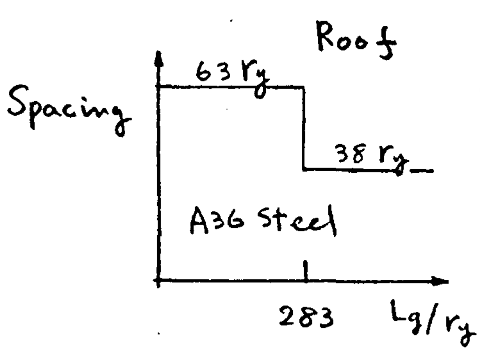
DESIGN EXAMPLE – PART3 WIND RESISTANT BENT B GIRDER LETERAL BRACING	TABLE 9. 32

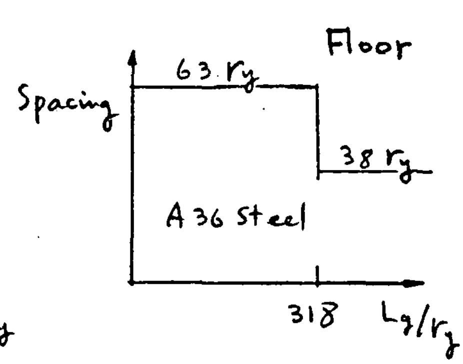
Joists are attached to the top flange of girders

(1)		(2)	(3)	(4)	(5)	(6)	(7)
LEVEL	SECTION	L_g ft	r_y in	$\frac{L_g \times 12'}{r_y}$	Max. Bracing Spacing Center	Max. Bracing Spacing Ends	Remarks
R	W10 x 15	11.0	.809	163		$63 r_y = 51$	OK
2	W12 x 19	11.0	.820	161		$63 r_y = 52$	OK
3	W14 x 22	11.0	1.04	127		$63 r_y = 65$	OK
R	W16 x 31	26.0	1.17	267		$63 r_y = 74$	OK
2	W18 x 35	26.0	1.23	253		$63 r_y = 77$	OK
3	W18 x 35	26.0	1.23	253		$63 r_y = 77$	OK

No additional bracing required







DESIGN EXAMPLE - PART 3 SUPPORTED BENT A GIRDER SHEAR	TABLES 9.33

	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
LEVEL	SECTION	d in	w in	V_u (1) Kips	1.7 W_L Klf	L_g ft	V_{max} (2) Kips	Remarks
R	W14×38	14 1/8	5/16	87.5	3.13	26.0	40.7	OK
R	W8×13	8	1/4	39.6	3.09	11.0	17.0	OK
2	W14×38	14 1/8	5/16	87.5	3.09	26.0	40.2	OK
2	W10×15	10	1/4	49.5	5.19	11.0	28.5	OK

1) $V_u = 0.55\ F_y w d$

2) $V_{max} = \dfrac{F w_L L_g}{2}$

DESIGN EXAMPLE - PART 3 WIND RESISTANT BENT B GIRDER SHEAR	TABLE 9.34

Exterior Girders ($L_g = 26.0$ ft)

LEVEL	SECTION	d in	w in	V_u 1) Kips	V 2) (F=1.7) Kips	V 3) (F=1.3) Kips	V_{max} 4) Kips	Remarks
R	W 16 x 31	15 7/8	1/4	78.7	40.7	31.8	40.7	OK
4	W 18 x 40	17 7/8	5/16	111	41.4	38.1	41.4	OK
12	W 24 x 61	23 3/4	7/16	205	41.4	53.6	53.6	OK
20	W 27 x 84	26 3/4	7/16	231	41.4	69.1	69.1	OK

1) $V_u = 0.55 F_y w d$

2) $W_L = 1.84 \times 1.7 = 3.13$ k/f (Roof)
$W_L = 1.88 \times 1.7 = 3.19$ " (Floor) } From Tab. 9.9(1)(5)

3) [Tab. 9.14(1) + Tab. 9.14(2)] / L_g + 31.1 (Roof)
[Tab. 9.14(1) + Tab. 9.14(2)] / L_g + 31.6 (Floor)

4) $V_{max}^{4)} = \max(V^{2)}, V^{3)})$

Interior Girders ($L_g = 11.0$ ft)

LEVEL	SECTION	d in	w in	V_u 1) Kips	V 5) (F=1.7) Kips	V 6) (F=1.3) Kips	V_{max} 7) Kips	Remarks
R	W 10 x 15	10	1/4	49.5	17.0	14.5	17.0	OK
4	W 14 x 26	13 7/8	1/4	68.7	29.2	36.9	36.9	OK
12	W 24 x 55	20 3/4	3/8	175	29.2	71.3	71.3	OK
20	W 24 x 84	24 1/8	1/2	239	29.2	105.8	105.8	OK

5) $W_L = 1.82 \times 1.7 = 3.09$ k/f (Roof)
$W_L = 3.13 \times 1.7 = 5.32$ k/f (Floor) } From Tab. 9.9(1)(5)

6) [Tab. 9.14(5) + Tab. 9.14(6)] / L_g + 13.0 (Roof)
[Tab. 9.14(5) + Tab. 9.14(6)] / L_g + 22.3 (Floor)

7) $V_{max}^{7)} = \max(V^{5)}, V^{6)})$

DESIGN EXAMPLE TYPICAL CONNECTIONS GIRDER TO COLUMN – WIND RESISTANT BENT B	TABLE 9.35

INTERIOR CONNECTION OF LEVEL 12

a. Moment resistant connection

b. Assume M_p of girder developed by flange plates and factored shear carried by web plate.

c. Flange plates for girders are shop-welded and one shear angle is shop-bolted to column. Other shear angle is bolted with 3/4" φ machine bolts for shipping.

d. Bolts in bearing-type connection are used.

Ultimate single shear strength of 7/8" A325-X bolts is

$1.7 \times 22 \times 0.6013 = 22.5$ Kip/bolt

Hole – 15/16" φ

e. Yield strength of material – A36

$F_y = 36$ ksi — Tension and compression
$F_p = 1.7 \times 1.35 \times 36 = 82.7$ kips — Bearing
$\tau_y = F_y/\sqrt{3} = 20.8$ ksi — Shear

f. Determine flange plate and bolts for girder.

<u>W24×61</u> $M_p = 422$ K-ft

$$C = T = \frac{M_p}{d_g} = \frac{422 \times 12}{23.75} = 213 \text{ kips}$$

Plate size – assume 7" width

$$t\,(7 - 2 \times 15/16) \times 36 = 213$$

$t = 1.15"$ Use $1\frac{1}{8}"$

No. of bolts $= \frac{213}{22.5} = 9.5$ – Use 10 – 7/8" A325-X

Bearing on flange plate

$10 \times 1.125 \times 0.875 \times 82.7 = 817 > 213$ kips <u>OK</u>

<u>W24×55</u> $M_p = 384$ K-ft

$$C = T = \frac{M_p}{d_g} = \frac{384 \times 12}{23.5} = 196 \text{ kips}$$

Plate size, Use $7'' \times 1\frac{1}{8}''$

No. of bolts $= \frac{196}{22.5} = 8.7$ Use 10-7/8" A325-X

g: Determine web connection

<u>W24x61</u> $V = 53.6$ Kips

No. of bolts $= \frac{53.6}{2 \times 22.5} = 1.19$ Use 2-7/8" A325-X

Bearing on girder web.

$2 \times 0.437 \times 0.875 \times 82.7 = 63.1 > 53.6$ <u>OK</u>

Bearing on angles, Assume 2Ls - 4 x 3½ x ¼ x 0'7"

$4 \times 0.25 \times 0.875 \times 82.7 = 72.2 > 53.6$ <u>OK</u>

Shear on angles

$2 \times 0.25 \times 7.0 \times 20.8 = 73.0 > 53.6$ <u>OK</u>

<u>W24x55</u> $V = 71.3$ Kips

No. of bolts $= \frac{71.3}{2 \times 22.5} = 1.59$ Use 2-7/8" A325-X

Bearing on girder

$63.1 < 71.3$ N.G. Use 3-7/8" A325-X

Bearing on angles, Assume 2Ls - 4 x 3½ x ¼ x 0'10"

$6 \times 0.25 \times 0.875 \times 82.7 = 108 > 71.3$ <u>OK</u>

Shear on angles

$2 \times 0.25 \times 10.0 \times 20.8 = 104 > 71.3$ <u>OK</u>

h: Check column web crippling at W24x61 flange plate

Eq. (8.1), $0.75 \times (0.561 + 5 \times 1.937) \times 36$ vs. $7 \times 0.561 \times 36$

$278 > 142$ <u>OK</u>

Eq. (8.4), $1.25 > 0.4\sqrt{7.0 \times 9/16} = 0.792$ <u>OK</u>

DESIGN EXAMPLE TYPICAL CONNECTIONS GIRDER TO COLUMN - WIND RESISTANT BENT B	TABLE 9.35 Cont'd

No stiffener req'd

i : Check column web for shear stress

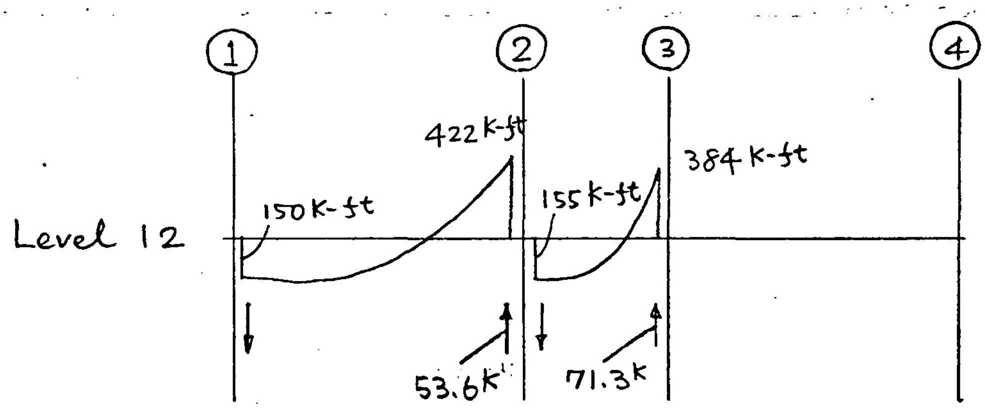
Eq. (8.9), $T_A = 213$ kips , $T_B = 155/23.5 = 66.0$

$$V_L = \frac{2 \times 238 \times 12}{(9'8'' - 24'')} = 68.0 \text{ kips} \quad \text{(Tab. 9.23 (4))}$$

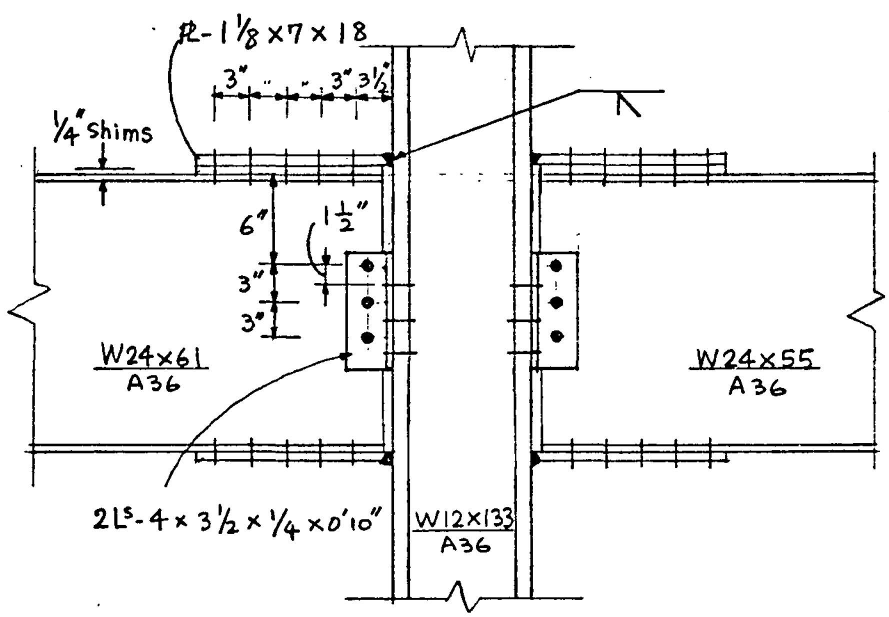
$$13.375 \times 0.75 \times 20.8 \quad vs \quad 213 + 66 - 68$$

$$209 \doteqdot 212$$

No stiffener req'd.



Level 12



REFERENCES

1. Okten, Omer S., Morino, Shosuke, Daniels, J. Hartley, and Lu, Le-Wu "EFFECTIVE COLUMN LENGTH FRAME STABILITY." Fritz Engineering Laboratory Report 375.2, Lehigh University, November 1973.

2. McNamee, Bernard M. "THE GENERAL BEHAVIOR AND STRENGTH OF UNBRACED MULTI-STORY FRAMES UNDER GRAVITY LOADING." PhD. Dissertation, Lehigh University, 1967.

3. McNamee, Bernard M. and Lu, Le-Wu "INELASTIC MULTISTORY FRAME BUCKLING." Proceedings ASCE, Vol. 98, ST 7, P. 1613, 1972.

4. ASCE "PLASTIC DESIGN IN STEEL." Manual No. 41, ASCE, New York, N. Y., 1971.

5. Daniels, J. H. COMBINED LOAD ANALYSIS OF UNBRACED FRAMES, Ph.D. Dissertation, Lehigh University, 1967, University Microfilms, Inc., Ann Arbor, Michigan

6. Daniels, J. H. A PLASTIC METHOD FOR UNBRACED FRAME DESIGN, Engineering Journal, AISC, Vol. 3, No. 4, October 1966, P. 141-149.

7. Driscoll, G. C., Jr., et al. PLASTIC DESIGN OF MULTI-STORY FRAMES - LECTURE NOTES, Fritz Engineering Laboratory Report 273.20, Lehigh University, 1965.

8. Parikh, B. P., Daniels, J. H., and Lu. L. W. PLASTIC DESIGN OF MULTI-STORY FRAMES -- DESIGN AIDS, Fritz Engineering Laboratory Report 273.24, Lehigh University, 1965.

9. Daniels, J. H. and Lu, L. W. THE SUBASSEMBLAGE METHOD OF DESIGNING UNBRACED MULTI-STORY FRAMES, Fritz Engineering Laboratory Report No. 273.37, Lehigh University, March 1966.

10. Daniels, J. H. and Lu, L. W. DESIGN CHARTS FOR THE SUBASSEMBLAGE METHOD OF DESIGNING UNBRACED MULTI-STORY FRAMES, Fritz Engineering Laboratory Report No. 273.54, Lehigh University, November, 1966.

11. Driscoll, G. C. , Jr., and Armacost, J. O. THE COMPUTER ANALYSIS OF UNBRACED MULTI-STORY FRAMES, Fritz Engineering Laboratory Report No. 345.5, Lehigh University, 1968, Document AD670 742, Clearing house.

REFERENCES (continued)

12. American Iron and Steel Institute, "PLASTIC DESIGN OF UNBRACED MULTISTORY STEEL FRAMES", New York, New York 1968.

13. Driscoll, G. C., Jr., Armacost, James O., III, and Hansell, W. C., "PLASTIC DESIGN OF MULTISTORY FRAMES BY COMPUTER", J. of the Struct. Div., ASCE, Vol. 96, STI, Jan. 1970

14. U.S.A. Standard Institute, "MINIMUM DESIGN LOADS IN BUILDINGS AND OTHER STRUCTURES", American Standard Building Code A58.1 - 1955.

15. Huang, J. S., Chen, W. F. and Beedle, L. S., "BEHAVIOR AND DESIGN OF STEEL BEAM-TO-COLUMN MOMENT CONNECTIONS," WRC Bulletin, 188, Oct. 1973

16. Chen, W. F. and Newlin, D. E., "COLUMN WEB STRENGTH IN STEEL BEAM-TO-COLUMN CONNECTIONS," Journal of Struct. Div., ASCE, Vol. 99, ST9, September 1973

17. Fisher, John W. and Struik, John H. A., "GUIDE TO DESIGN CRITERIA FOR BOLTED AND RIVETED JOINTS," Wiley-Interscience

18. Blodgett, O. W.
DESIGN OF WELDED STRUCTURES, The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio 1966.

REFERENCES

AND

these quotes are not needed, since the title is capitalized

1. Okten, Omer S., Morino, Shosuke, Daniels, J. Hartley, and Lu, Le-Wu "EFFECTIVE COLUMN LENGTH/FRAME STABILITY," Fritz Engineering Laboratory Report 375.2, Lehigh University, November 1973.

2. McNamee, Bernard M.
"THE GENERAL BEHAVIOR AND STRENGTH OF UNBRACED MULTI-STORY FRAMES UNDER GRAVITY LOADING," PhD. Dissertation, Lehigh University, 1967.

O.K.

P

3. McNamee, Bernard M. and Lu, Le-Wu
"INELASTIC MULTISTORY FRAME BUCKLING," Proceedings ASCE, Vol. 98, ST 7, P. 1613, 1972.

no blanks

4. ASCE
"PLASTIC DESIGN IN STEEL," Manual No. 41, ASCE, New York, N. Y., 1971.

5. Daniels, J. H.
COMBINED LOAD ANALYSIS OF UNBRACED FRAMES, Ph.D. Dissertation, Lehigh University, 1967, University Microfilms, Inc., Ann Arbor, Michigan

6. Daniels, J. H.
A PLASTIC METHOD FOR UNBRACED FRAME DESIGN, Engineering Journal, AISC, Vol. 3, No. 4, October 1966, P. 141-149.

7. Driscoll, G. C., Jr., et al.
PLASTIC DESIGN OF MULTI-STORY FRAMES - LECTURE NOTES, Fritz Engineering Laboratory Report 273.20, Lehigh University, 1965.

8. Parikh, B. P., Daniels, J. H., and Lu. L. W.
PLASTIC DESIGN OF MULTI-STORY FRAMES — DESIGN AIDS, Fritz Engineering Laboratory Report 273.24, Lehigh University, 1965.

9. Daniels, J. H. and Lu, L. W.
THE SUBASSEMBLAGE METHOD OF DESIGNING UNBRACED MULTI-STORY FRAMES, Fritz Engineering Laboratory Report No. 273.37, Lehigh University, March 1966.

10. Daniels, J. H. and Lu, L. W.
DESIGN CHARTS FOR THE SUBASSEMBLAGE METHOD OF DESIGNING UNBRACED MULTI-STORY FRAMES, Fritz Engineering Laboratory Report No. 273.54, Lehigh University, November, 1966.

11. Driscoll, G. C. , Jr., and Armacost, J. O.
THE COMPUTER ANALYSIS OF UNBRACED MULTI-STORY FRAMES, Fritz Engineering Laboratory Report No. 345.5, Lehigh University, 1968, Document AD670 742, Clearing house.

REFERENCES (continued)

12. American Iron and Steel Institute, PLASTIC DESIGN OF UNBRACED MULTISTORY STEEL FRAMES, New York, New York, 1968.

13. Driscoll, G. C., Jr., Armacost, James O., III, and Hansell, W. C., PLASTIC DESIGN OF MULTISTORY FRAMES BY COMPUTER, J. of the Struct. Div., ASCE, Vol. 96, ~~ST1~~, Jan. 1970. "one"

14. U.S.A. Standard Institute, MINIMUM DESIGN LOADS IN BUILDINGS AND OTHER STRUCTURES, American Standard Building Code A58.1 - 1955. start the next line

15. Huang, J. S., Chen, W. F. and Beedle, L. S., BEHAVIOR AND DESIGN OF STEEL BEAM-TO-COLUMN MOMENT CONNECTIONS, WRC Bulletin, 188, Oct. 1973.

16. Chen, W. F. and Newlin, D. E., COLUMN WEB STRENGTH IN STEEL BEAM-TO-COLUMN CONNECTIONS, Journal of Struct. Div., ASCE, Vol. 99, ST9, September 1973.

17. Fisher, John W. and Struik, John H. A., GUIDE TO DESIGN CRITERIA FOR BOLTED AND RIVETED JOINTS, ~~Wiley-Interscience~~ John Wiley and Sons, 1974.

18. Blodgett, O. W.
DESIGN OF WELDED STRUCTURES, The James F. Lincoln Arc Welding Foundation, Cleveland, Ohio, 1966.