Seismic Design Practice for Steel Buildings

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TO PROPERLY INTERPRET seismic codes and their revisions, it is becoming increasingly important that the design engineer have an understanding of the basic theory behind seismic code provisions. This paper provides a brief treatment of the subject of seismic theory and design, particularly as it applies to structural steel. Much of the theory is condensed into simple terms more readily applied to the typical problems faced by busy design engineers. Specific seismic code provisions are discussed, to aid in their interpretation.

The author has specifically addressed the situation of engineers designing for the earthquake problems and building codes of the State of California. No attempt has been made to cover conditions not applicable to California. However, it is hoped that the paper will prove helpful to any engineer involved in the seismic design of steel buildings.

SCOPE

Sect. 1: Seismic Design Terminology (Part 1)—Basic definitions needed to support the explanation of the code equivalent static load method of seismic design. By separating these definitions out of the code theory discussion, closer continuity is maintained in explanations of the code provisions and a more accessible terminology reference is provided.

Sect. 2: Basis for 1974 SEAOC Seismic Code—Discussion of the equivalent static load concept as defined in the 1974 SEAOC Code,* because almost all of the code provisions, and design under this code, are confined to this method of seismic design.

Sect. 3: Seismic Design of a 7-Story Office Building—Application of the basic code provisions to the seismic design of a steel frame. It should be remembered that the code provides minimum design requirements and the calculations provided here are intended to show how these minimum provisions apply. It is beyond the scope of this paper to comment on the adequacy of the code minimums in relation to any particular level of damage risk.

Sect. 4: Building Code Variations from the 1974 SEAOC Seismic Code—Discussion of the principal building code variation from the SEAOC Code, the requirement of a dynamic design for special buildings. This requirement is not often applied at the present time, but this is definitely the direction which future revisions to the SEAOC Code will take. Engineers are urged to become familiar with this method of seismic design in order to better evaluate their present designs and future code proposals.

Sect. 5: Seismic Design Terminology (Part 2)—Definitions for an extended list of seismic terms that are most often encountered in the requirements or criteria for a dynamic seismic design. This section is intended to support the explanation of dynamic design given in Sect. 4.

Sect. 6: Drift Control Analysis for Steel Moment Frames—Calculations and factors involved in the drift control of steel moment frames. This subject is considered important enough to merit thorough coverage because moment frame drift components have had little in-depth coverage, despite the fact that distortion control is really the most important consideration in seismic design.

Appendix A: SEAOC Code, Partial Text of Earthquake Regulations—Section 1 of the 1974 SEAOC Code; also a cross-reference for these regulations and formula numbering as used in the 1976 Uniform Building Code.

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**Although the Structural Engineers Association of California publishes a "Recommended Lateral Force Requirements and Commentary" (1974), rather than a code, the recommendations are more commonly referred to as the SEAOC Code. For simplicity and clarity the more common terminology is used in this paper. Somewhat the same logic applies to the use of the term "seismic" to imply "aseismic."*

Subjects Not Included—Many topics related to seismic design are not covered in the above listed sections. Specifically excluded are the following topics:

A History of Seismic Code Development—This paper is primarily intended to aid in the understanding and application of the 1974 SEAOC Code. The most important point to recognize concerning seismic code development is the fact that code minimums have generally increased with time as the understanding of the potentials for earthquake damage has increased. It is almost certain that today's bare code minimum building will not satisfy tomorrow's code.

Mathematical Derivation of Dynamic Response Formulas—Many texts present the mathematics related to simple dynamic systems. There seems to be little need to repeat this material. The mathematical solution for complex dynamic systems is a whole field in itself.

The Cause and Prediction of Earthquakes—Statistical records of past earthquake ground motions and mathematical extensions of these records to predict future ground motions are deemed to be of more immediate concern to the practicing structural engineer than their cause and close time prediction.

Seismic Design Recommendations—It should not be inferred from this paper that a seismic design to exact code minimums is a recommended design. However, since any recommendations for a design which is more conservative than code minimums must be somewhat subjective, such recommendations have generally been avoided. It is pointed out, however, that a fine line design (or analysis) which goes to great lengths to comply (or show compliance) with code minimums certainly misinterprets the basis and spirit of the code. If the exact unique earthquake ground motion to which a building will be subjected were known, such attempts at exact numerical results might be justified. Since, at best, only the general range of possible relevant ground motions is known, it is general conservative accuracy, rather than close numerical accuracy, which is needed.

GENERAL DISCUSSION

The following points have general application throughout the paper.

Code seismic provisions for steel frame buildings, and most of the discussions in this paper concerning the theory behind those provisions, apply to all buildings, short or tall, whether they have shear walls, braced frames, or moment frame lateral force resisting systems. It may seem that more emphasis is placed on tall buildings and moment frames, since dynamic data often refer to tall buildings, and a separate section (Sect. 6) is included on the drift of moment frames. However, any emphasis on tall buildings is a matter of available data, since tall buildings, generally being more dynamically regular and also being more instrumented, are the source of more data. The separate section dealing only with the drift control of moment frames just reflects the fact that drift is almost always critical to seismic moment frame design, whereas drift is seldom critical to shear wall or to braced frame design.

The vibratory response of buildings to ground motions is essentially elastic, except for those time intervals when the motion is violent enough to cause significant inelastic yielding in the structure. The study of dynamic response is, therefore, almost entirely related to elastic vibration. Throughout this paper the word "elastic", whether included or not, should be inferred unless inelastic behavior is specifically noted.

The SEAOC Code provisions, and these discussions, may seem to imply that response forces due to lateral ground motions apply to only one axis at a time. Horizontal ground motions are not unidirectional. For any given site there will be ground motions oriented radially to the source event and perpendicular to those radial motions. Components of these motions, of course, can be resolved for the two building axes. The code addresses itself to only one of those components at a time. It is tacitly assumed that amplification of the forces in a building due to simultaneous vibration on the two axes can be generally represented by some amplification of the uniaxial forces.

The code does not require design consideration of vertical ground motions. This is because the design for vertical loads includes safety factors which will generally provide for the forces due to vertical ground motions. It is recognized that this premise may not apply when large overturning forces are involved, but this factor is generally included in the total probability of occurrence picture, rather than as a separate variable.

NOMENCLATURE

A_w	Cross-sectional area of column or girder web
$*C$	Code lateral force coefficient, used with other factors in base shear formula
C_1	Lateral force coefficient equal to V/W
$*C_p$	Lateral force coefficient for portion of structure
C_T	Period mode shape constant
C'	Lateral force coefficient equal to $S_v(2\pi/g)$
$*D$	Building plan dimension in direction parallel to force
F	Lateral force
$*F_t$	Lateral force at top of structure
$*F_x$	Lateral force at any level
f	Natural frequency of vibration
g	Acceleration of gravity
h	Frame height (c. to c. girders)

**SEAOC Code terms.*

$*h_n$	Building height to level n [code formula (1-3A)]
h_c	Clear height of column
H	Building height above rigid base
$*I$	Importance factor related to occupancy used in lateral force formula
$*K$	Frame constant used in lateral force formula
l	Girder length (c. to c. columns)
l_c	Girder clear length between columns
m	Mass
P	Vertical load on column
$P\Delta$	Frame moment due to laterally displaced vertical load
$*S$	Site structure coefficient
S_a	Spectral (max.) response acceleration
S_v	Spectral (max.) response velocity
S_d	Spectral (max.) response displacement
$*T_s$	Characteristic site period
$*T$	Period of vibration for SDF system; fundamental (first mode) period for MDF system
$*V$	Lateral force or shear at base of structure
$*Z$	Seismicity zone factor used in lateral force formula
SDF	Single degree of freedom system
MDF	Multi-degree of freedom system
ω	Circular frequency
λ	Stiffness constant
Δ	Lateral displacement (at top of structure unless noted otherwise)
Δ_c	Lateral displacement due to column distortion
Δ_g	Lateral displacement due to girder distortion
Δ_j	Lateral displacement due to joint distortion

SECT. 1: SEISMIC DESIGN TERMINOLOGY (PART 1)

The key to understanding a subject is knowledge of the vocabulary involved. The following general definitions of common terms used in seismic design are presented as an aid to the reader. Simplicity and clarity are given precedence over exactitude in these definitions.

Not covered in this section are the terms defined in the code, since these terms are legally defined and only the code interpretive bodies may interpret the definitions. Terms involved only in the more extensive dynamic analyses are also omitted from this section. More terms are included in Sect. 5 to support the discussion of dynamic analysis and design methods.

Dynamic Forces (as they relate to earthquakes)—Briefly, all parts of a structure and its contents exert static vertical loads on the structure due to their stationary dead weight, but they also exert *dynamic* loads due to inertia forces if they are in motion. Static loading is independent of the supporting structure. Dynamic loading is entirely dependent on the dynamic characteristics of the supporting structure. Static loads are independent of loads preceding the loads being considered. Dynamic loads vary with every change in motion and, at any given time, generally depend on the preceding motion as well as the motion at the instant considered.

Dynamic Response—Every structure will vibrate in accordance with the laws of harmonic motion as determined by its own dynamic characteristics. The dynamic characteristics are a function of its weight and stiffness. A building's response to the motion of its base is determined by those dynamic characteristics.

Period of Vibration—The period of vibration T is the time necessary to complete one cycle of oscillation and is the reciprocal of the *natural* frequency of vibration f. The *natural* frequency is equal to the *circular* frequency ω divided by 2π. The circular frequency of a single degree of freedom structure is proportional to the square root of the stiffness divided by the mass. The equations are:

$$\omega = \sqrt{\lambda/m}, \text{ where } \lambda = \text{stiffness and } m = \text{mass}$$

$$f = \omega/2\pi$$

$$T = 1/f = 2\pi/\omega = 2\pi\sqrt{m/\lambda}$$

Expressing mass as W/g and stiffness as F/Δ (force over deflection),

$$T = 2\pi\sqrt{\frac{W\Delta}{gF}}$$

This is the formula for a single-degree of freedom (SDF) system. If F is expressed as C_1W, the formula can be expressed as:

$$T = \frac{2\pi}{\sqrt{g}}\sqrt{\frac{\Delta}{C_1}} = 0.32\sqrt{\frac{\Delta}{C_1}}$$

where Δ is expressed in inches.

Dynamic Systems—An SDF system has one lumped mass and can be represented by a single weight mounted on top of a slender weightless vertical cantilever rod. This system will have a period of vibration dependent on the stiffness of the rod and the size of the weight on top. A series of such rods, of different height but with the same rod cross section and top weight, will illustrate a spectrum of periods because the stiffnesses will vary with the length of the rod.

A structure with several lumped masses is a multi-degree of freedom (MDF) system and its vibration will be a combination of the vibrations due to the several lumped masses. There will be as many *modes* of vibration as there are lumped masses. Each mode has its own period and can be represented by an SDF system of the same period. The mode with the longest period is called the *first*, or *fundamental*, mode and the modes with shorter periods (higher frequencies) are called the *higher* modes. The period relations depend on the mode shapes.

Mode Shapes—The deflected shape of a structure for any single mode of vibration is always the same for that structure, regardless of the magnitude of the vibration. In other words, though the amplitude of the displacement changes with time, the relation between displacements throughout the height remains constant. The distribution of accelerations for a single mode of vibration therefore remains constant. Knowing the mode shape and the maximum vibration at the top, the maximum vibration at any level above the base can be directly obtained for that mode.

Figure 1.1 shows mode shapes for a building with at least 5 floors (lumped masses). The modal displacements (S_1, S_2, etc.) are not normally drawn to any actual scale relationship with each other. For a typical smoothed response spectrum they will decrease as the modal period decreases from the lower to the higher modes.

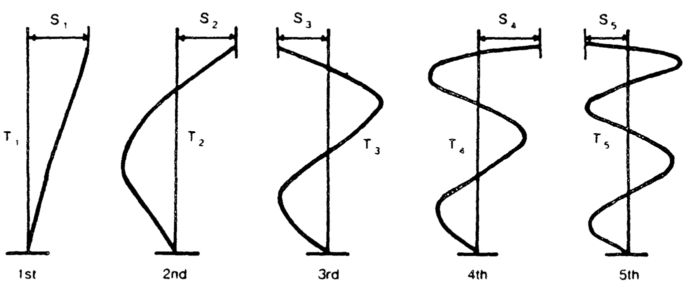


Fig. 1.1. Mode shapes

Shear Beam/Bending Beam—The dominant deflected shape of a building as it responds to the ground motion at its base is often related to that of a "shear beam" or a "bending beam." Those terms refer to the dominant drift components of the building as a whole, acting as a single cantilever member. The deflected shape implied by the bending (or chord) drift component is clearly given by a line whose slope *increases* continuously from its base to its top. However, the deflected shape implied by the shear drift component depends entirely on the shear stiffness distribution, so that the term "shear beam" needs qualification. A uniform mass, uniform section cantilever beam will have a slope which continually *decreases* from its base to its top. A uniform mass, tapered section cantilever beam might have a uniform slope from its base to its top. A "shear beam" may even have an "S" shape if the stiffness vs. lateral force relation varies in this manner.

Response Spectra (Elastic)—The vibration of an SDF system due to a continuously varying base motion will, at any time, be the summation of the effects of the base motion impulses to that time. The maximum vibration reached during any length of time after the base motion starts is its spectral (maximum) value. If a series of SDF systems is subjected to the same base motion, there will be a series of maximum values related to SDF system periods, which will form a spectral curve for that base motion. Thus, any given irregular motion will produce an individual response curve or response spectrum. Knowing the base motion and SDF period, the maximum vibration at the top of an SDF system can be picked off the appropriate curve, measured in terms of acceleration, velocity, or displacement relative to the base.

MDF Elastic Response Spectra—If any fixed relation between the periods of the modes of vibration of an MDF system is assumed, the computer can be programmed to obtain the response of all of the considered modes simultaneously. The individual modal responses are modified by the participation factors related to the given modal relationship. The computer then adds the responses algebraically at each time interval of the input ground motion and records the maximum algebraic response. This process can be repeated for an entire series of first mode periods, each with its related higher modes, until a complete MDF response spectrum for any structure with the given modal relationship is obtained for the given input motion. MDF spectra for the most typical modal period relationship (1, $1/3$, $1/5$, $1/7$, etc.) have been run for many of the key design earthquake motions.

Damping—A perfectly elastic system, set into vibratory motion, would continue to vibrate forever if the vibration were not stopped by an outside force. However, no system is perfectly elastic, and the vibratory motion will die out due to loss of energy resulting from internal strains. This loss of energy is called *damping*. Damping is generally expressed as a percentage of "critical damping," the damping which would stop the vibratory motion in one swing after free vibration starts. The first small percentages of damping greatly reduce peak responses because peak responses are generally associated with short response time durations and therefore involve little energy. Damping represents energy losses from many sources and therefore can be of a number of types as related to vibration.

Base Shear—This is the total horizontal seismic shear at the base of a structure and is a function of the acceleration of each of the masses of the structure relative to the base, the effects added algebraically at any instant. For a static design the base shear is determined as an assumed relative acceleration times the total mass of the structure (force equals mass times acceleration).

Shear Distribution—If a building is assumed to vibrate predominantly in its fundamental mode, with the deflection curve a straight line (uniform drift), the amplitude of vibration is proportional to the height. Since the fundamental period of vibration applies to the whole building, the acceleration at any level must also be proportional to its height above the base. The shear will then vary linearly from zero at the base to a maximum at the top. The triangular distribution of shear in the Code is based on this general assumption. Some confusion enters here because the word shear is used to represent a force at a floor and also the sum of the forces down to that floor. The triangular loading pattern and the shape of the building shear envelope due to that loading are, of course, very different. See Fig. 1.2.

Figure 1.2

The triangular loading is sometimes spoken of as "throwing weight to the top." Actually, it is a matter of assuming higher accelerations at the top. For most buildings of any significant height-to-width ratio, the acceleration varies more than linearly, due to the effect of the higher modes. Statistically, this can be represented simply by placing a portion of the total base shear at the roof and distributing the rest triangularly.

Overturning Moment (Rocking Moment)[2]—The overturning moment is the algebraic sum of the moments of all the forces above the base multiplied by their heights above the base. If the forces are represented by an envelope of maximums which are reached at different times, then the overturning moments will be overestimated. However, they are not greatly overestimated, since the first mode is dominant for these moments and the forces for the first mode do all reach an algebraic maximum at the same time.

It should be recognized that overturning moments are almost never a threat to overturn buildings, because the transitory nature of the loading does not allow time enough for the building to move past its center of rotation. However, this type of relief can not be assumed to help much in regard to the generation of axial forces in columns or frame rocking due to insufficient frame dead load or tie-down resistance.

Story Drift Coefficient—The story drift coefficient is the ratio of inter-story horizontal displacement to story height, usually expressed as a percent. Thus, for a 12 ft (144 in.) story height, a 1% drift coefficient indicates a story drift of 1.44 in. For that portion of the drift due to joint rotation, the coefficient measures the tangent of the rotation angle. Since the angle is small, it is also a direct measure of the rotation angle itself (in radians).

Ductility—The term *ductility* refers to the ability of an assembly to withstand considerable distortion after yielding without a great loss of strength. This is an important property in regard to all kinds of loading, since it determines the ability of an assembly to adjust local elastic load distribution to prevent sudden failure and allow more total load to be imposed before failure. For seismic loading, the property is doubly important because it allows a structure to absorb energy and withstand more intense ground motions after its yield strength is exceeded. In other words, for static loading, if an assembly is ductile enough, the yield strength of any one element can be exceeded but failure will not occur until the entire assembly reaches yield capacity. In the case of dynamic loading, failure will not even occur after the entire assembly reaches yield, as long as $P\Delta$ moments are not excessive; however, the ductility may have to be maintained through a number of small inelastic strain reversals and a few large strain reversals.

SECT. 2: BASIS FOR 1974 SEAOC SEISMIC CODE

Response Formula—The basic code "static equivalent" seismic response formula is

$$C = 1/15T^{1/2} = 0.067/\sqrt{T} \leqslant 0.12$$

This formula is in the form of a generalized response spectrum based on the following assumptions and theory.

Earthquake ground motions, since they are generated in, and are transmitted through, complex ground structures, are erratic, random, and constantly changing throughout the duration of strong motion. Determining the dynamic response of even one SDF system to this erratic motion is a complex mathematical problem. However, modern computers can be programmed to quickly determine and plot the entire time record of the response. In fact, the computer can quickly determine the entire record of the response for a large number of different SDF systems. This is, however, an immense volume of data, so it is reduced to simplify the record of the maximum response reached at any time during the earthquake for each SDF system. This, then, produces a spectrum of maximum responses (spectral acceleration, velocity, or displacement) to a given ground motion sequence for a range of dynamic systems represented by their dynamic periods of vibration.

Since seismic vibration response is essentially harmonic motion, spectral acceleration S_a, spectral velocity S_v, and spectral displacement S_d are related for any individual SDF system by the following equations:

$$S_a = \frac{2\pi}{T} S_v \qquad S_v = \frac{2\pi}{T} S_d \qquad S_a = \frac{4\pi^2}{T^2} S_d$$

$$S_v = \frac{T}{2\pi} S_a \qquad S_d = \frac{T}{2\pi} S_v \qquad S_d = \frac{T^2}{4\pi^2} S_a$$

Each individual site ground motion will have its own unique response spectrum. For that ground motion, the maximum response (S_a, S_v, or S_d) of each individual SDF system can be picked off the spectrum, and the effect of change in period can be exactly determined. The spectrum for an erratic, random ground motion is an erratic response curve, however, so that the effect on response of small changes in period is also erratic and does not necessarily indicate a general trend for other ground motions. To obtain a design spectrum, it is necessary to study the spectra for a great number of representative ground motions and a number of statistically generated motions, and to draw a smooth curve which essentially envelopes all of the spectra. An envelope curve, of course, does not show the true relationship between response and system-period for any given ground motion, but does *provide* for each motion response within the bounds of the envelope. Only those responses which reach the envelope at some point are critical to the development of the envelope.

An examination of a great many response spectra based on experience in the State of California indicates the following general pattern:

1. None of the spectra show a true resonant response, such as that generated by the simple regular motion of mechanical equipment. This eliminates concern that the response amplification can approach an uncontrolled resonance.
2. Different ground motions will have spectral accelerations, S_a, which peak at different periods. However, for the typical California critical ground motions, the peak will occur for some system whose period is in the 0.2 to 0.5 sec. range. The peak S_a for a 10% damped system will be on the order of 2 to 2.5 times the maximum ground acceleration. The S_a for SDF systems whose periods are less than 0.2 sec. will generally reduce from the peak S_a as the period reduces. An infinitely stiff system ($T = 0$) would obviously have the same acceleration as the ground.
3. For an SDF system whose period is greater than that of the system which has the peak S_a, the value of S_a decreases rapidly as the period increases. This decrease is generally studied by examining the spectral velocity S_v since, being the integral of the acceleration, it is less affected by peak accelerations of short duration and it is a somewhat smoother curve. The averaging envelope value of spectral velocities is seen to be roughly a constant for periods greater than the period at peak acceleration. Referring back to the vibration equations, this is seen to indicate that S_a reduces from peak acceleration approximately as the inverse of the period increase.

For MDF systems, this information cannot be used directly. A structure with several lumped masses (such as floors, in buildings) is a multi-degree of freedom (MDF) system and its vibration will be a combination of the modes of vibration of the several lumped masses. The period of the modes decreases rapidly from the fundamental (first) mode, and for typical buildings the decrease is of the order 1, $1/3$, $1/5$, $1/7$, $1/9$, etc. The participation (in effect, the percent of the total building mass acting in that mode) decreases even more rapidly than the modal periods, so that only the first few modes are generally significant.[1] Therefore, for most smoothed spectra, and particularly for an envelope spectrum, the fundamental mode dominates.

Since the fundamental mode of vibration generally dominates, the MDF system spectral acceleration (S_a) curve (generally given in terms of percent of gravity acceleration or base shear coefficient) generally follows the form of the SDF system spectra. The essential differences from the SDF characteristics are in the very low period range and for periods over 1.0 sec. For very low periods, the higher mode periods, which are a fraction of the fundamental period, fall into the range of responses which are declining from the peak as the period approaches zero. Therefore, the response is essentially that of the fundamental mode, but with a participation factor reduction. Peak MDF acceleration responses are therefore less than SDF responses. For periods greater than about 1.0 sec., the higher mode periods may add to the response and MDF response may start to exceed the SDF response.

It can be seen from the SDF response discussion that the purely elastic response of an SDF system, for a range of periods with a constant spectral velocity, is given by the formula $a/g = C'/T$, where $C' = S_v(2\pi/g)$. This formula would apply over the period range where spectral velocity S_v is a constant value, or up to the peak acceleration period.

For a uniform mass, uniform drift coefficient building, the purely elastic MDF response, assuming a constant spectral velocity, is given by the formula $a/g = C'/T^{3/4}$. If it is assumed that the spectral velocity value in the design range is increasing somewhat with period, rather than a constant value, the exponent for the period T would be less than $3/4$. The code formula, which uses an exponent of $1/2$ for T, could reflect this assumption or could represent a subjective empirical design response adjustment. The effect of decreasing the exponent is to decrease the formula response for shorter building periods and increase the response values for longer periods.

Since the code response formula, $C = 1/15T^{1/2}$, is used for code stress design, with modifiers and working stresses, the constant in the formula does not directly relate to the response to any given level of ground motion intensity.

The spectra shown in Figs. 2.1 through 2.6 are for strong earthquakes represented by the El Centro 1940 record and five simulated records. They illustrate how an envelope of spectra can be developed to show the trend relations of acceleration, velocity, and displacement to building period. MDF spectra for a typical modal period relationship are shown with the standard SDF spectra for purposes of comparison. Note the change in scale from Fig. 2.1 to Fig. 2.2 and the multiplier in Fig. 2.4 if amplitudes are being compared.

SINGLE DEGREE OF FREEDOM ABSOLUTE ACCELERATION SPECTRA
5% DAMPING

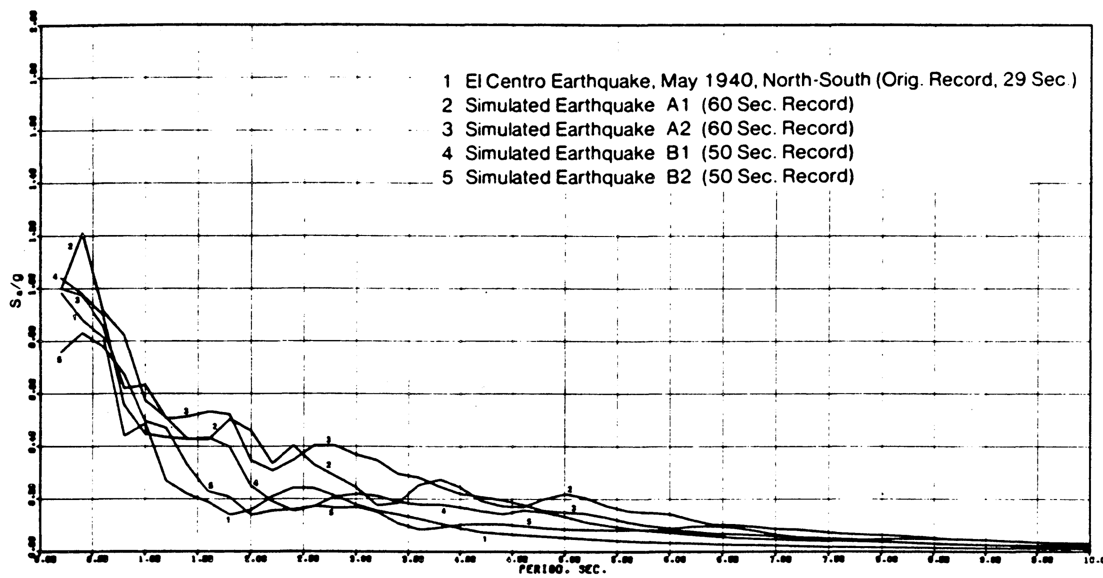


Figure 2.1

MULTI-DEGREE OF FREEDOM BASE SHEAR COEFFICIENT SPECTRA
Computed For 8 Modes, 5% Damping
Modal Period Relations 1, 1/3, 1/5, 1/7, 1/9, 1/11, 1/13, 1/15

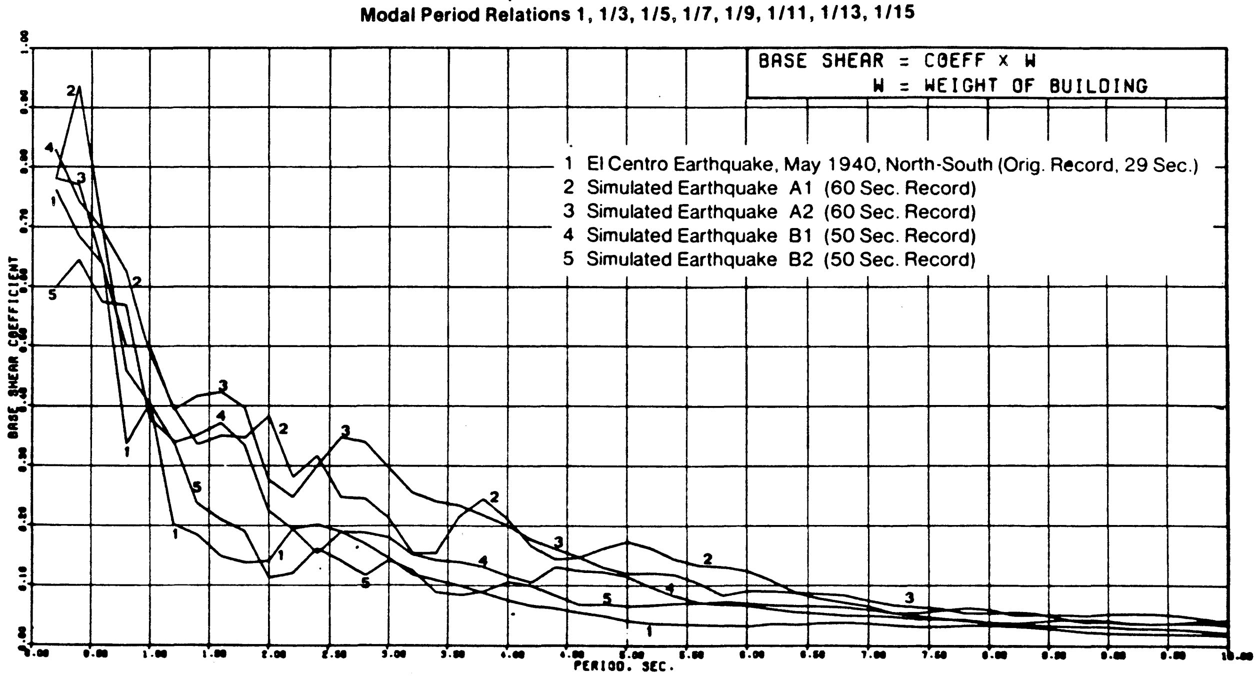


Figure 2.2

SINGLE DEGREE OF FREEDOM VELOCITY RESPONSE SPECTRA
5% Damping

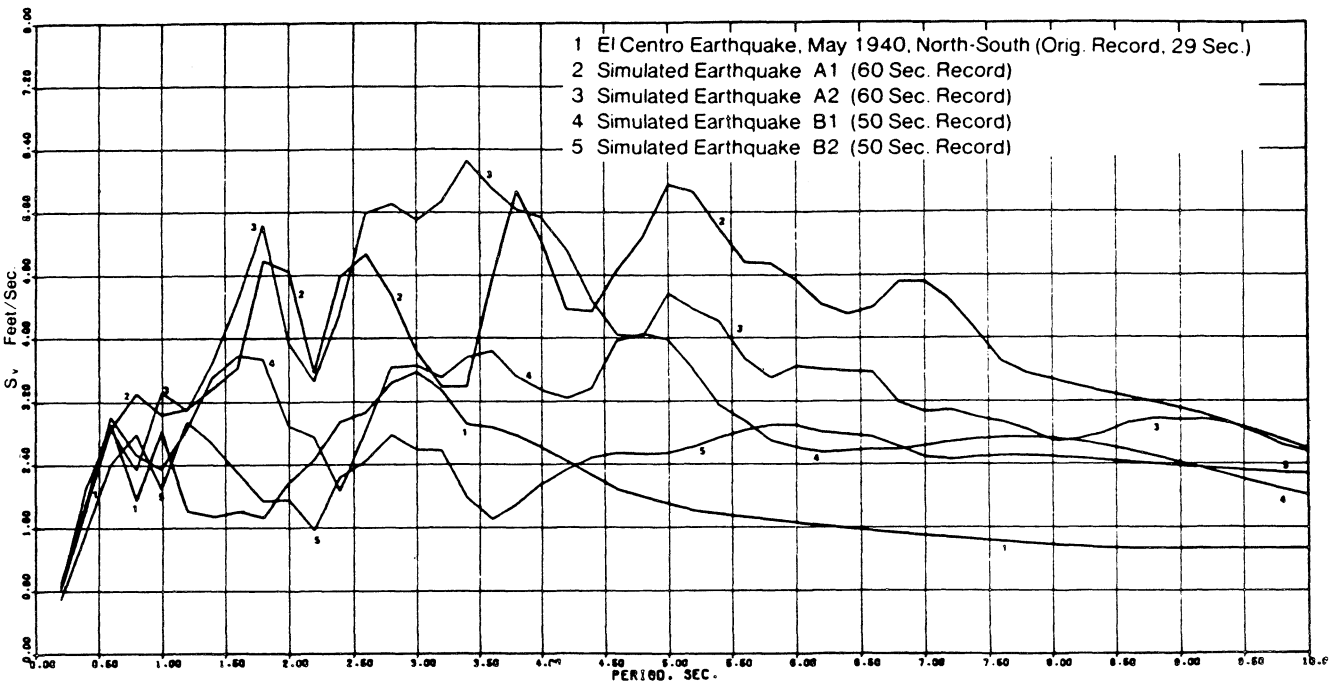


Figure 2.3

MULTI-DEGREE OF FREEDOM VELOCITY RESPONSE SPECTRA
Computed For 8 Modes, 5% Damping
Modal Period Relations 1, 1/3, 1/5, 1/7, 1/9, 1/11, 1/13, 1/15

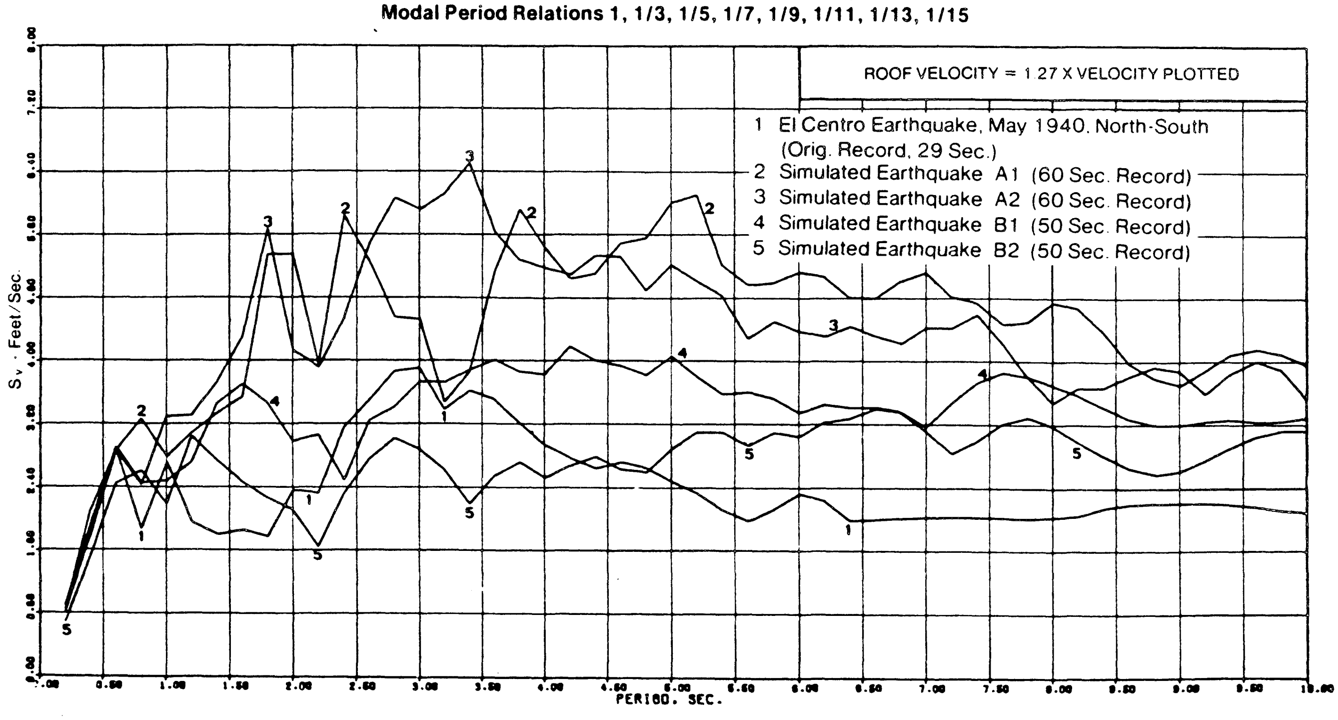


Figure 2.4

SINGLE DEGREE OF FREEDOM RELATIVE DISPLACEMENT SPECTRA
5% Damping

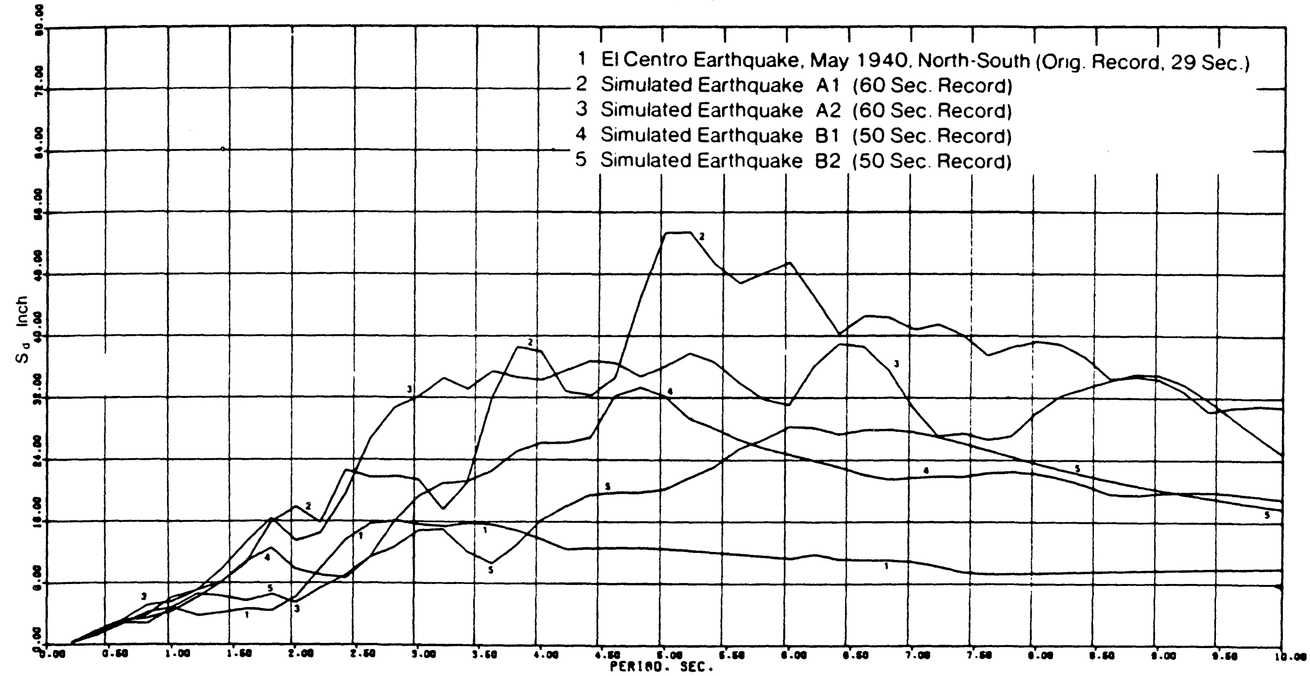


Figure 2.5

MULTI DEGREE OF FREEDOM RELATIVE ROOF DISPLACEMENT SPECTRA
Computed For 8 Modes, 5% Damping
Modal Period Relations 1, 1/3, 1/5, 1/7, 1/9, 1/11, 1/13, 1/15

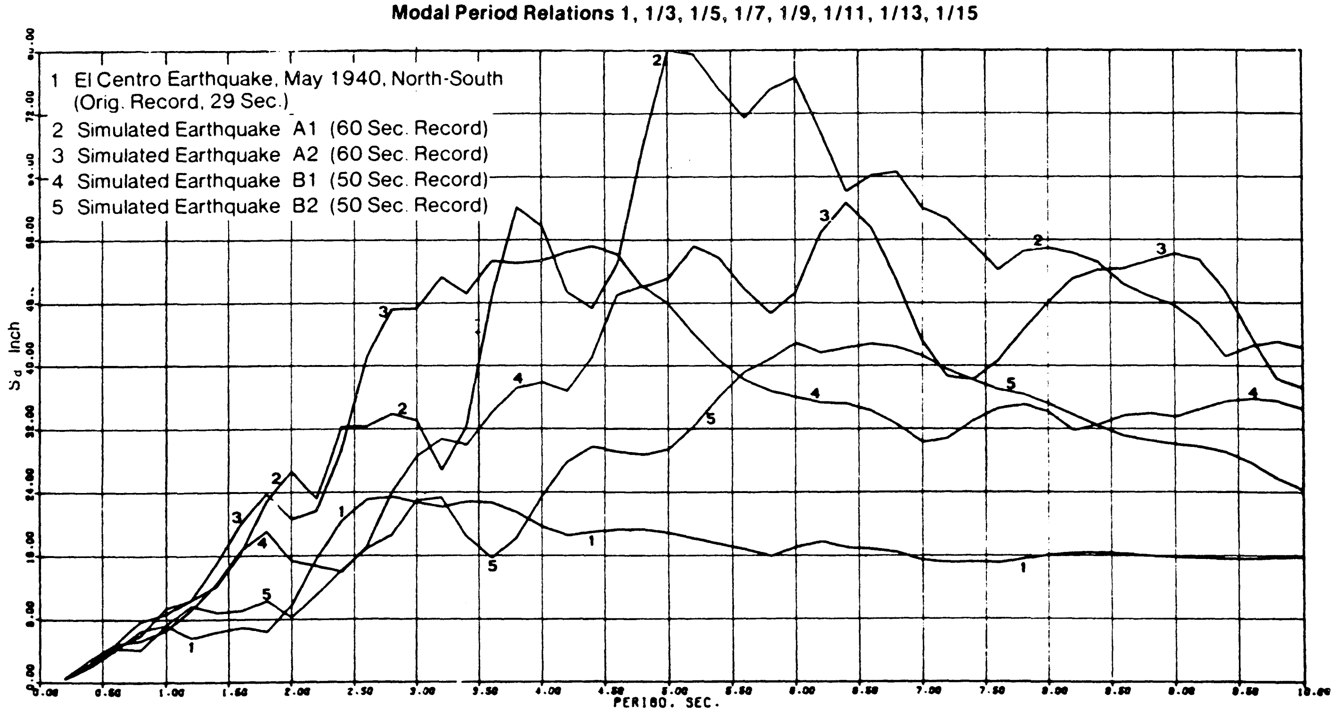


Figure 2.6

Base Shear Formula—The code base shear formula, $V = ZIKSCW$, includes four modifiers to the response formula C-value. These are empirical factors established by code committees and are explained in the code (see Appendix A.) The modifier S assumes a direct relation between the site period, the ground motion which may be expected to be generated at the site, and the response of different period buildings at that site.

Figure 2.7 shows graphically how the code S-factor relates to the quantity T/T_s. Figure 2.8 shows the minimum and maximum S-factors related to building period and the code limits put on assumed T and T_s values. Figure 2.9 shows the resulting minimum and maximum CS-values related to building period. These plots may be useful as a code design aid, since CS and T are interdependent and cannot be solved for directly.

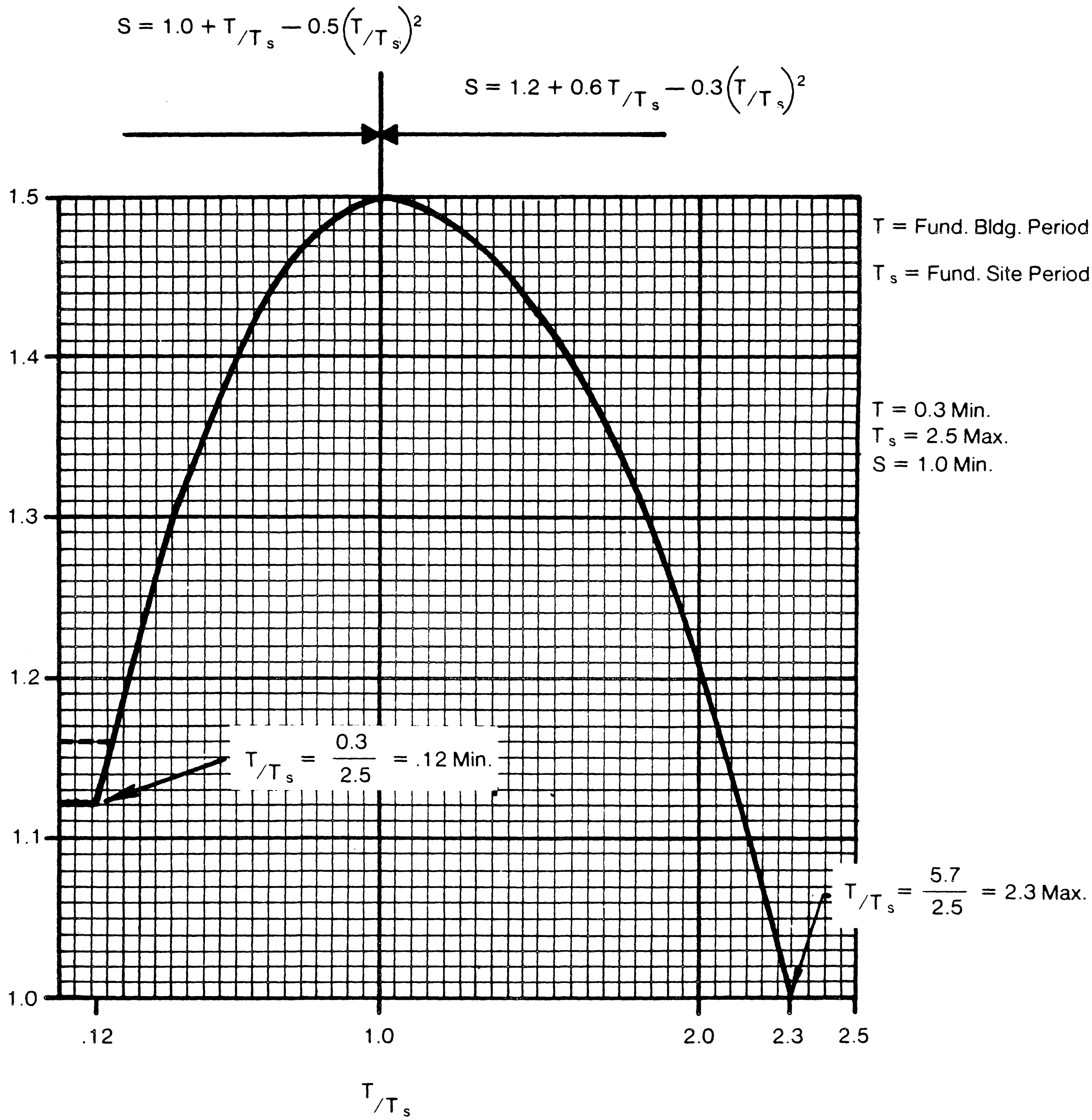


Fig. 2.7. S factor related to T/T_s

Building Period Formulas—Building periods are directly related to the square root of the horizontal displacement at the roof caused by a dynamic horizontal force equal to the weight of the building above the base. Since this is a *dynamic* force, the total horizontal force due to the weight of the building must be distributed throughout the height of the building in direct proportion to the variation of dynamic response acceleration. The acceleration, relative to the base, increases approximately uniformly with the height for most buildings; this is the reason for the triangular distribution of shear force in the SEAOC Code, with the maximum lateral force at the top. This is the basis of the code Formula (1-3), which will closely approximate the fundamental (first) mode period of a building when the design drift is known.

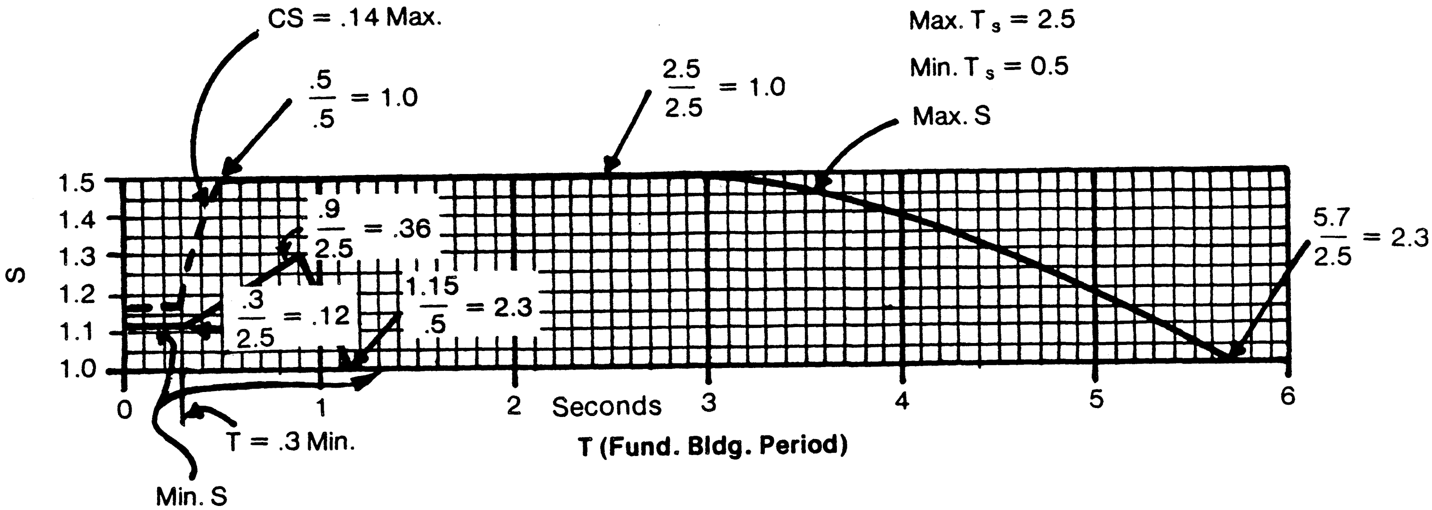


Fig. 2.8. S factor related to building period

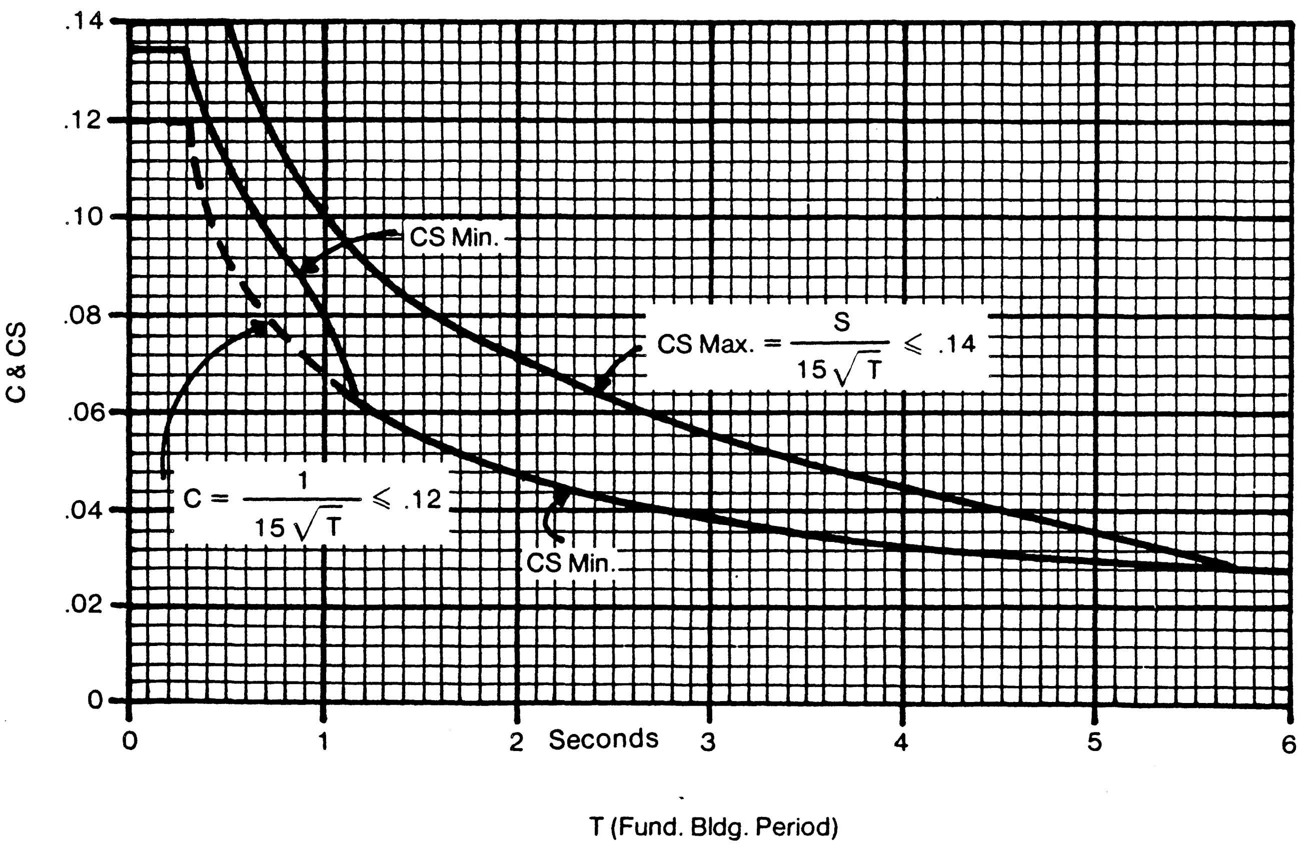


Fig. 2.9. C and CS factors related to building period

However, a much simpler formula can be obtained by noting that the basic period formula is $T = C_T\sqrt{\Delta/C_1}$, where Δ is the horizontal displacement at the top, derived as noted. The term C_1 is the force coefficient used to derive Δ (equal to 1 if the total building weight is used as the horizontal dynamic force, or *ZICS* if the code drift force coefficient is used). The period constant C_T depends on the deflected shape, but can be closely approximated for the typical MDF constant drift building. The formula $T = 0.25\sqrt{\Delta/C_1}$ (Δ in inches) gives a close period estimate for the typical MDF building.[2] This is a simplified version of code Formula (1-3) and one which can also be used to closely approximate the period when only the drift coefficient or proposed drift coefficient is known.

The code formula $T = 0.10N$ implies that, for a constant drift building, the period varies directly with the roof displacement. Since T must vary as the square root of the roof displacement, this period relation would only be true if the constant drift were computed for a force which decreases directly with N. A straight line period variation, such as given by this formula, can give reasonably good results for only a limited building height range. This formula yields reasonable period estimates for buildings in the 40-story range, but very poor estimates for short buildings.

Code Formula (1-3A), $T = 0.05h_n/\sqrt{D}$, is an empirical formula based on recorded periods for shear wall buildings. For these buildings it will seldom be necessary or feasible to compute a really accurate drift. The formula seems to give as close an estimate for T as could be derived empirically for this type of building.

Shear Distribution—Code Formula (1-5) separates out of the base shear (total horizontal seismic force on the building) a portion (F_t) to be applied directly at the top level. This portion is given by Formula (1-6).

The remainder of the base shear is distributed to all of the levels above the base, including the roof, according to Formula (1-7). This formula provides the basic triangular distribution of forces which is related to a uniform dynamic stiffness, uniform dynamic drift building. Moderate variations of the triangular distribution, which are caused by dynamic stiffness variations related to mass distribution, are accounted for by including the actual weight at each level and the height above the base for each level.

The envelope of maximum dynamic shears obtained by time-history dynamic computer analyses for many buildings subjected mathematically to many earthquake ground motions confirms this general MDF envelope distribution of horizontal forces.

The loading and shear distribution given by the code formulas are illustrated in Fig. 2.10. It can be seen from the shear diagram that the new code formula for F_t, which reasonably approximates the true dynamic force distribution, has a marked effect on design only at the top, where the frame is light, and on the overturning (rocking) moment.

Figure 2.10

Structural Feature Provisions—Paragraphs 1(E)2 through 1(E)5 of the SEAOC Code require consideration of several structural features that affect dynamic response, though without giving much specific design direction. This is partly because the many variables are difficult to codify in static force equivalents, and partly because much of the research needed to establish limiting parameters has not been accomplished. These paragraphs will be discussed generally here, but their interpretation may rest with the building department involved.

Setbacks [Paragraph 1(E)2]—There has been much discussion on this subject, at the time of writing the 1960 code and up until the 1974 revision. The original provision was really aimed at a building tower whose plan size was reduced near the top; however, by far the most common application is the office tower with a large low base.

It has been shown by dynamic computer analysis (Fig. 2.11) that a low rigid shear wall base supporting a flexible moment frame tower will cause a tower response which is little changed from the tower response if the tower were set on the ground. On the other hand, the response of a greatly reduced tower section near the top of the building may be greatly amplified because of the plan size reduction. The first question to be answered is whether the change in building dynamic stiffness is great enough to cause the building to act as two systems, a primary and a secondary system. If the building does act as two systems, the base system may pass on the ground motion to the secondary system with little effective change. This is certainly true with a low rigid base. A flexible base system, however, may filter out some of the high frequency motion and provide an amplified and more simple harmonic motion at the base of the secondary system. This type of base motion may result in an amplified response for the secondary system.

The code draws the line at 75% of the base plan dimension, but does not specify how the response is ob-

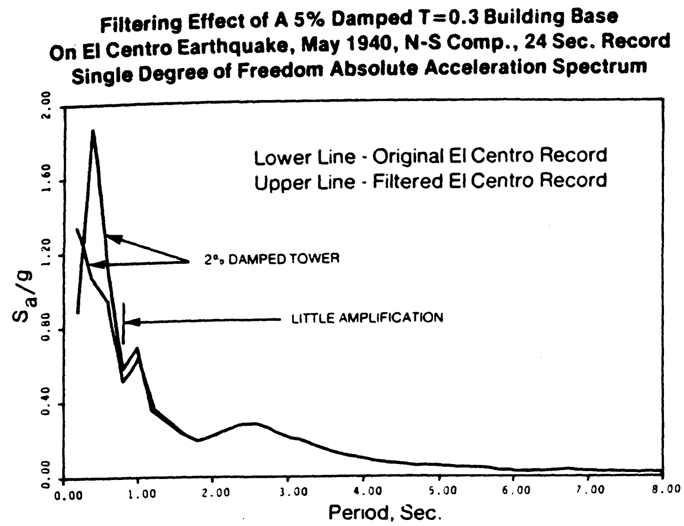


Figure 2.11

tained for greater setbacks. The need for some kind of dynamic analysis or analogy is pointed out.

Figure 2.11 shows the effect of a rigid base on the response of a building supported on it.

Dynamically Irregular Buildings [Paragraph 1(E)3]—This paragraph is really an extension of the paragraph on setbacks to include all types of highly irregular dynamic features. It appears that a dynamic analysis or analogy could be used to relate the design forces, on a static basis, to the equivalent static force requirements for a dynamically regular building.

Distribution of Horizontal Shear [Paragraph 1(E)4]—It is specified that the horizontal shear force applied at any level be distributed to the vertical shear resisting elements in proportion to their rigidities. Consideration of the stiffness of the horizontal bracing is required, but usually this system is very stiff in comparison with the vertical elements. It is not specified that the rigidity of the story vertical bracing system consider the stiffness of the vertical system below. If there are significant bending drift (vertical axial deformation) differences between two systems sharing a common horizontal force, it is obvious that shear stiffness comparisons for an isolated story will not show a true force distribution.

Rigid elements which may affect the performance of the designed frame, or which may be affected by the frame distortion, are required to be "considered and provided for in the design." The primary concern for the effect of these elements on the frame is that they may induce an increased dynamic response which adds critical axial loads due to overturning moments, or may change the yield sequence in a moment frame from moment to shear. The primary concern for the effect of the frame on non-frame elements is that the elements may not be able to accommodate to the distortion forced upon them without losing the capacity to support *themselves.* Not spelled out in the code, but critical to design, is the differentiation between the capability of a non-frame element to distort without loss of load-carrying capacity, and the capability of the element to hang on to the frame without serious loose fragmentation. An element may "fail" in shear or moment at a given distortion without losing its capacity to hang on and hang together. With adequate distortion control by the structural frame, this is usually assured if code connection requirements are met.

Stiffnesses are always based on elastic deflections, whereas the resistance to large response forces is generally a mixture of elastic and inelastic stiffnesses. Some judgment should be used in determining to what extent a very detailed and "exact" elastic distribution of shears is valid and necessary. Not only relative, but actually expected, deformations need to be considered.

Horizontal Torsional Moments [Paragraph 1(E)5]—Shear distribution to vertical elements is affected by the horizontal torsional moment due to eccentricity between the center of mass and the center of vertical element stiffnesses. The code requires that the actual eccentricity be considered and that not less than 5% be assumed, even for the common very regular building. For the general case, the effect is to provide a slightly higher lateral design force, since the code requires that negative torsional shears be neglected. The provision for neglecting negative shears is conservative, but may defeat a rational solution.

Torsion introduces other modes of vibration which may couple with the translational modes to give an amplified response. There is no simple way to determine torsional period or the combining of responses, however, and the code therefore relegates this problem to the highly irregular building category requiring special dynamic consideration. Post-earthquake computer dynamic investigations of buildings which have been subjected to earthquakes have not found this to be a dominant problem.[9,10]

Overturning Forces—The code specifies, in a general way, that an analysis be made for the axial forces induced by the code design horizontal forces, distributed according to code throughout the height of the building, as the building cantilevers from its base. A rational stiffness related distribution of horizontal forces and their induced vertical forces is required all the way down to the foundations.

The code specifies that "every structure shall be designed to resist the overturning effects." It is generally assumed that the "effects" of uplift must be resisted by dead loads. Factoring of forces and dead loads is left to the basic building codes.

Lateral Force on Elements of Structures—Under this heading are included elements which are attached to the structural frame, rather than elements of the frame itself. These attached elements respond dynamically to the motion of the frame, and not to the motion of the ground. Since the frame has fixed modal periods, resonance between the attached element and the frame may be a problem. The C_p-values given in the code are an attempt to give static force equivalence for different elements, considering their possible response and the consequences involved if they collapse or fall.

Drift Provisions—The 1974 SEAOC Code has introduced a story drift coefficient limit of 0.005 (0.5%). In computing the drift, the force formula K-factor reduction is eliminated, since this factor is intended to reflect the ability of a given type of frame to control forces rather than displacements. A ductile moment frame has a K-factor force reduction of 0.67, because it is assumed that it can displace after yielding without *capacity* failure. Inelastic analyses indicate that it will *displace* about the same as if the force built up without yielding.

The code does not presume that a frame designed to 0.5% drift at code forces will be limited to 0.5% drift for strong earthquakes. Nor does the code assume that 0.5% drift is the maximum tolerable drift. It must be presumed that a frame designed to this limit will have 0.5% drift during a moderate earthquake shaking and on the order of 1.5% for very strong shaking. The code implies this by requiring that the effects of drifts equal to 3 times the 0.5% limit for frames (at code lateral forces) be provided for in regard to non-frame elements. Computer dynamic analyses of buildings for their predicted response to strong ground motions support this general order of drift prediction.

Structural Systems

Ductility Requirements—Only paragraphs 1(J)1d, e, and g need comment. Considerations for *non-frame* elements were discussed under the Shear Distribution paragraph of the Structural Features section. Paragraphs d and e apply specifically to vertical *frame* elements which are not part of the frames designed to resist code seismic forces. These elements must continue to resist their design vertical loads when the frame distorts to a drift $3/K$ times that for code forces. They are not required to have any lateral force resistance capacity, but must maintain the required vertical load capacity.

Paragraph g specifies that braced frames shall be designed to 1.25 times the force given in the basic force formula (1-1). Connections must be designed to a load factor of (1.25 × 1.33) or 1.67, since the 33% increase in working loads is deleted for braced frame connections. The code provides an alternate of developing the full capacity of the braced frame members at the connections but, since the full capacity is not easily defined, increasing the force by an arbitrary 33% will probably be more common. The object of the force increase for braced frames is, as for shear walls, to account for the lower assurance of adequate ductile inelastic energy absorption in these systems.

Special Requirements: Exterior Elements [Paragraph 1(J).3d]—Exterior elements attached to structural frames present a special hazard to people below. The connections of rigid elements to frames must accommodate the maximum frame distortion or they will be peeled off, regardless of their strength. The Code specifies that a drift $3/K$ times that at code forces must be accommodated by the connections. Connections must be ductile and must attach to imbedded reinforcement in concrete or must be designed to accommodate sliding in the joinery (slotted bolt holes).

Steel Ductile Moment Resisting Frames—The SEAOC requirements are best explained by quoting from the Commentary written at the time that the ductility provisions were adopted.

Definitions—"The *joint* is the entire assemblage at the intersections of the numbers. In a tier building, this consists of many elements such as the web panel within the joint, the stiffeners, if any, that form the continuation of the beam flanges, the column flanges within the joint, and the external connection material.

"The *connection* consists of *only* those elements that connect the member to the joint. In the case of the beam welded to column flanges, the connection consists only of the welds and the erection clips that are provided."

Connections—"The original (1960) SEAOC Code has no provisions concerning frame joints. As a result, the connections could be designed using working stress design to meet the stresses resulting from prescribed lateral forces. This could result in frames which would not have the ductile behavior required. To provide frame ductility it is essential that connections must be strong enough to force a plastic hinge in the weakest connected member, or the connection must be able to deform plastically without serious loss of resistance.

"The simplest and most common means of satisfying this requirement is to make the connection of the beam to the column capable of developing the full plastic capacity of the connected beam. The new provisions therefore require this. The exception would allow the requirement to be modified if it can be shown that inelastic joint displacement would take place without a hinge forming in the beam. This could take place if a hinge were to form at the end of the column, a hinge were to form in the web panel of the joint, or the connection itself were capable of sufficient inelastic deformation to form a competent hinge."

Local Buckling—"Plastic design research has determined thickness limitations which will insure development of the full plastic moment before local buckling. It is not necessary to develop the full plastic moment as long as the required moment is maintained throughout the required deformation. However, compliance with "plastic design sections" requirements provides hinge development. Should research demonstrate the reliability of other sections such as thin web welded girders or trusses, the Code can be modified to incorporate the necessary requirements."

Slenderness Ratios (Related to Sidesway)—This section of the commentary no longer applies because the requirement it covered (that even shear wall or braced frame buildings be considered unbraced, for the purpose of determining column slenderness ratio stress reductions) has been deleted. However, columns in moment frame buildings without shear walls or braced frames must still comply with the building code allowable stress reduction formulas related to slenderness ratios for unbraced frames. This complicates the design of these columns if the columns are sized to just comply with the allowable stresses. However, since under the drift limit requirement of the 1974 SEAOC Code stiffness will generally govern much of the design of moment frames, it is suggested that conservative column design to simplify design and aid in drift control is justified. Exact compliance with the formulas related to buckling of very flexible frames has doubtful relevance for frames designed for drift control under seismic forces.[3] It is suggested that compliance with stress reduction formulas be used as a check, rather than as design criteria. This design consideration is discussed further at the end of Sect. 6.

SECT. 3: SEISMIC DESIGN OF A 7-STORY STEEL BUILDING

General Design Information

Code and Design Criteria:

The building will be designed in accordance with the forthcoming 1976 Edition of the Uniform Building Code (UBC). Seismic design is based on Chapter 23 of the UBC, which is essentially the same as the 1974 SEAOC Code.* Design of steel members and connections is based on Chapter 27 of the UBC, whose applicable provisions for this design example are also contained in the 1973 UBC.

For symbols and notations not defined in this design example, see Appendix A and the UBC.

**The 1976 UBC seismic design formulas are identical with the 1974 SEAOC Code formulas. Both UBC and SEAOC formula numbers are included in Appendix A of this paper, where Section 1 of the SEAOC Code is reproduced for reader reference.*

The structure is an office building, Group F, Div. 2 occupancy, Chapter 11 of UBC, and Type 1 construction, as per Chapter 5 of UBC. Two-hour fire protection for floors and roof and three-hour for columns and girders are required as per UBC Table No. 17-A. This protection is provided by a spray-on type of fireproofing material.

The building is located in Seismic Zone No. 3. It is also in the vicinity of the San Andreas Fault; therefore, the seismic zone is designated as Zone No. 4. The engineering geologist has determined that the characteristic site period $T_s = 1.0$ sec.

The frame is to be structural steel. As shown in Fig. 3.1, it is braced in the N-S direction on column lines 1 and 5. Ductile moment frames are provided in the E-W direction, along column lines A and D. Floors and roof are 3-in. metal deck with 3¼-in. lightweight (110 pcf) concrete fill. Typical story height is 11 ft-6 in., based on 8 ft-0 in. clear ceiling height.

Material specifications are:

Steel frame: A36
High-strength bolts: A325-F
Welding electrodes: E70

Loads:

Roof Loading:	
Roofing and insulation	7.0 psf
Metal deck	3.0
Concrete fill	44.0
Ceiling and mechanical	5.0
Steel framing and fireproofing	8.0
Dead load	67.0 psf
Live load (reducible), UBC Sect. 2305(a)	20.0
Total load	87.0 psf

Floor Loading:	
Metal deck	3.0 psf
Concrete fill	44.0
Ceiling and mechanical	5.0
Partitions, UBC Sect. 2302(b)	20.0
Steel framing, incl. beams, girders, columns, and spray-on fireproofing	13.0
Dead load	85.0 psf
Live load (reducible), UBC Sect. 2304	50.0
Total load	135.0 psf

Curtain Wall:	
Average weight including column and spandrel covers	15.0 psf

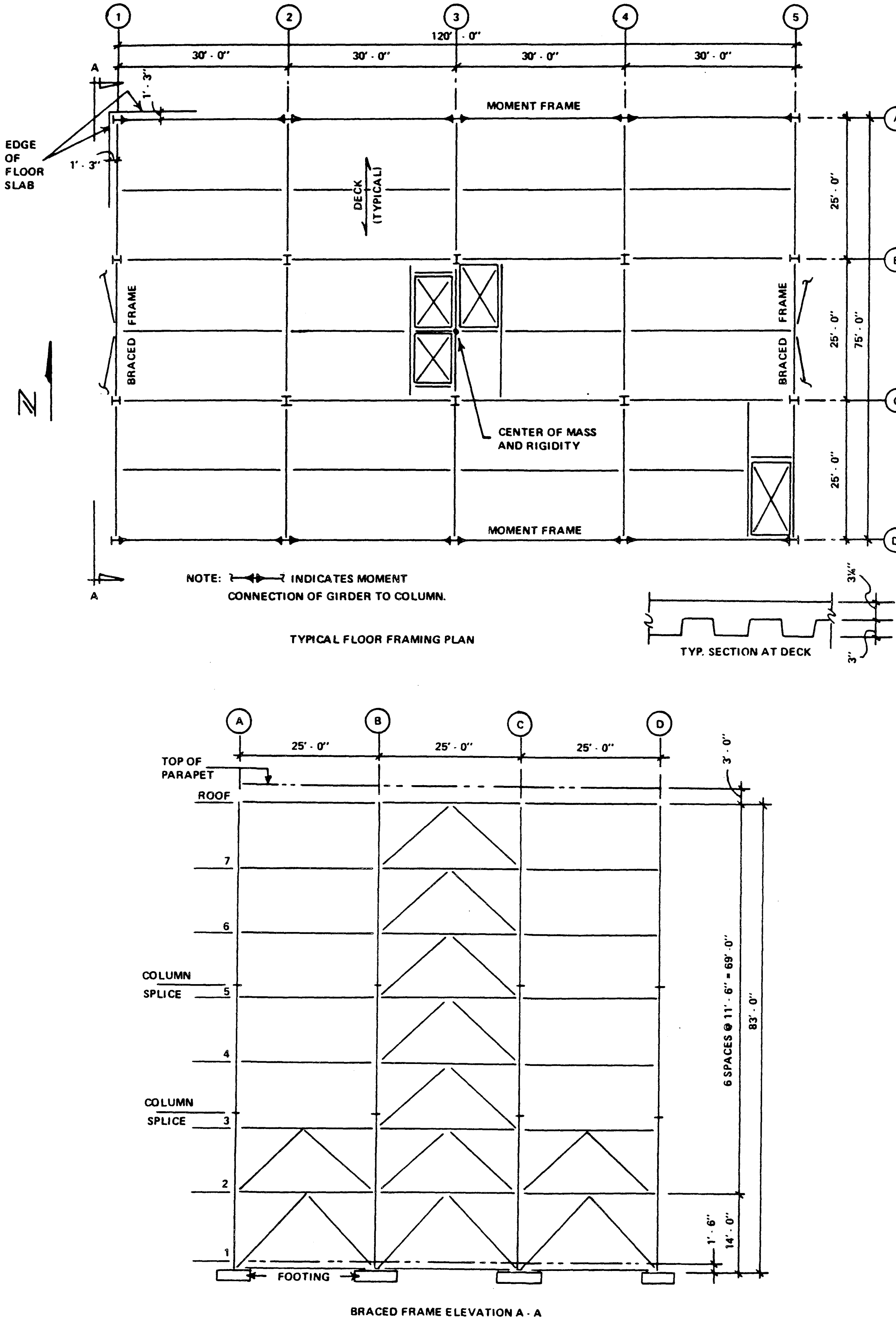


Fig. 3.1. Framing plan and braced frame elevation

Moment Frame Design

East-West Seismic Forces:

$V = ZIKCSW$ 1976 UBC Formula (14-1)

$Z = 1.00$ for Zone No. 4
$I = 1.00$ per UBC Table No. 23-K
$K = 0.67$ per UBC Table No. 23-I
$V = (1.00)(1.00)(0.67)(CSW) = 0.67CSW$

In order to determine C and S, the period T may be taken as:

$$T = 0.10N = 0.10(7) = 0.70 \text{ sec.} \qquad (14\text{-}3B)$$

However, use of this formula results in a poor estimate of the period for a relatively low, moment-resisting frame building. As previously stated in Sect. 2,* $T = 0.10N$ yields a reasonable period estimate for buildings in the 40-story range, but is not accurate for shorter buildings. Therefore, the basic period formula for contant drift (moment frame) buildings will be used:

$$T = 0.25\sqrt{\Delta/C_1}$$

where
T = Period of building, sec.
Δ = Lateral deflection at top of building, in.
C_1 = Lateral force coefficient by which the total weight of the building is multiplied in order to obtain the seismic lateral force due to the building's response to a given base motion

Drift limitations usually control the design of a moment frame and UBC Sect. 2314(h) limits the drift to 0.5%. Therefore,

$$\Delta = 0.005H = 0.005(83.0 \times 12) = 4.98 \text{ in.}$$

As related to the UBC:

$$C_1 = ZICS = (1.0)(1.0)(CS) = CS$$

Note that the factor K is omitted from the above equation, since it is a factor assigned to a type of construction in recognition of its inherent resistance to earthquakes and, therefore, is not directly related to stiffness and drift.

Substituting, $T = 0.25\sqrt{4.98/CS}$

Since both C and S are rather complex functions of T, the solution to this equation might be by trial and error. However, a more direct solution can be achieved by assuming a value for S, which has a rather narrow range of values: $1.0 \leqslant S \leqslant 1.5$. Assume $S = 1.0$.

$$C = \frac{1}{15\sqrt{T}} \qquad (14\text{-}2)$$

See the earlier discussion of Building Period Formulas, second and third paragraphs.

$$T = 0.25\sqrt{\frac{4.98}{(1/15\sqrt{T})(1.0)}}$$
$$= 0.25\sqrt{(4.98)(15)(T^{1/2})} = 2.16T^{1/4}$$
$$T^{3/4} = 2.16$$
$$T = (2.16)^{1.33} = 2.8 \text{ sec.}$$

Check the value of S:
Since T_s has been given as 1.0 sec.,

$$T/T_s = 2.8/1.0 = 2.8 > 1.0$$

$$S = 1.2 + 0.6(T/T_s) - 0.3(T/T_s)^2 \qquad (14\text{-}4A)$$
$$= 1.2 + 0.6(2.8) - 0.3(2.8)^2$$
$$= 0.53 < 1.0 \text{ (min. } S)$$

Therefore, the assumption of $S = 1.0$ is correct and T may be taken as 2.8 sec.

$$C = 1/15\sqrt{T} = 1/15\sqrt{2.8} = 0.04$$

$$W_{flr} = (122.5 \times 77.5)(0.085) + (400 \times 11.5)(0.015) = 874 \text{ kips}$$

$$W_{rf} = (122.5 \times 77.5)(0.067) + (400 \times 8.75)(0.015) = 687 \text{ kips}$$

$$W = 6(874) + 687 = 5{,}930 \text{ kips (total dead load)}$$

$$V = 0.67CSW = 0.67 \times 0.04 \times 1.0 \times 5930 = 160 \text{ kips (total lateral force)}$$

The total lateral force is distributed over the height of the building in accordance with UBC Formulas (14-5), (14-6) and (14-7). See Fig. 3.2.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (14\text{-}5)$$

$$F_t = 0.07TV \qquad (14\text{-}6)$$
$$= 0.07 \times 2.8 \times 160 = 31 \text{ kips}$$

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} = \frac{(129)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (14\text{-}7)$$

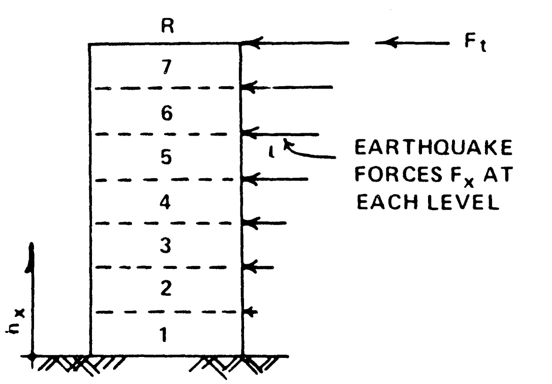


Fig. 3.2. Distribution of earthquake forces over height of building

The distribution of lateral forces over the height of the building is shown in Table 3-1, along with corresponding drift data. Since drift criteria will determine the size of the girders and columns of the moment frame, it is possible to verify the building period by UBC Formula (14-3):

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div g\left[\sum_{i=1}^{n-1} F_i\delta_i + (F_t + F_n)\delta_n\right]} \qquad (14\text{-}3)$$

$$= 2\pi\sqrt{404 \div (32.2)(74.8)} = 2\pi\sqrt{0.168}$$
$$= 2.6 \text{ sec.}$$

This value is in good agreement with the building period of 2.8 sec. previously determined from the basic period formula [Formula (14-2)]. Therefore, $T = 2.8$ sec. is a reasonably accurate period for this building in the east-west direction and there is no need to revise the calculations.

Wind loading was not critical for lateral forces in this design example; however, if wind should control the design of the frame, then it would be necessary to calculate both the period and the earthquake forces based on stiffness requirements of the frame to resist wind.

Distribution of Earthquake Forces:

Although the centers of mass and rigidity coincide, UBC Sect. 2314(e)5 requires designing for a minimum torsional eccentricity, e, equal to 5% of the maximum building dimension.

$$e = (0.05)(120) = 6.0 \text{ ft}$$

Both the moment frames and the braced frames will resist this torsion. Due to the braced frames being much stiffer than the moment frames, the relative rigidities are assumed as follows:

$$R_A = R_D = 1.00; \qquad R_1 = R_5 = 4.00$$

Shear Distribution in the E-W Direction:

$$V_{A,x} = R_A\left[\frac{V_x}{\Sigma R_{E\text{-}W}} \pm \frac{(V_x e)(d)}{\Sigma R_y(d)^2}\right] = V_{D,x}$$

where

d = Distance from frame to center of rigidity

$R_{E\text{-}W}$ = Rigidity of those frames extending in the east-west direction

R_y = Rigidity of a braced or moment frame, referred to that frame on column line y

V_x = Total earthquake shear on building at story x

$V_{y,x}$ = Earthquake shear on a braced or moment frame referred to that frame on column line y at story x

$$\Sigma R_{E\text{-}W} = 2(1.00) = 2.0$$

$$\Sigma R_y d^2 = 2(1.00)(37.5)^2 + 2(4.00)(60.00)^2 = 31{,}600$$

$$V_{A,x} = 1.00\left[\frac{V_x}{2.00} \pm \frac{(V_x \times 6.00)(37.5)}{31{,}600}\right]$$
$$= 1.00[0.50V_x \pm 0.007V_x]$$
$$= 0.51V_x = V_{D,x}$$

The second term (torsion) within the bracket being small indicates that the moment frames are resisting little torsion due to eccentricity.

Table 3-1

Floor Level	h_x (ft)	w_x (kips)	$w_xh_x \times 10^{-2}$	$\frac{w_xh_x}{\Sigma w_ih_i}$	F_x[a] (kips)	V_x[a] (kips)	F_x[b] (kips)	V_x[b] (kips)	δ_x[c] (ft)	$w_x\delta_x^2$	$F_x\delta_x$
R	83.0	687	570	0.203	(26+31)[d]	—	(39+47)[d]	—	0.41	115	35.2
7	71.5	874	625	0.222	29	57	43	86	0.36	113	15.5
6	60.0	874	524	0.187	24	86	36	129	0.30	79	10.8
5	48.5	874	424	0.151	19	110	28	165	0.24	50	6.7
4	37.0	874	324	0.115	15	129	23	193	0.18	28	4.1
3	25.5	874	222	0.079	10	144	15	216	0.13	15	1.9
2	14.0	874	122	0.043	6	154	9	231	0.07	4	0.6
1	—	—	—	—	—	160	—	240	—	—	—
Σ	—	—	2811	1.000	160	—	240	—	—	404	74.8

[a] Forces or shears for use in stress calculations of the frame.

[b] Forces or shears for stress multiplied by $1/K = 1.5$ to obtain corresponding values for drift calculations. See Sect. 2314(h) of the 1976 UBC (or Sect. 1(H) of the 1974 SEAOC Code, Appendix A of this paper).

[c] $\delta_x = 0.005h_x$. See references in footnote b.

[d] At roof, $F_x = (F_t + F_n)$.

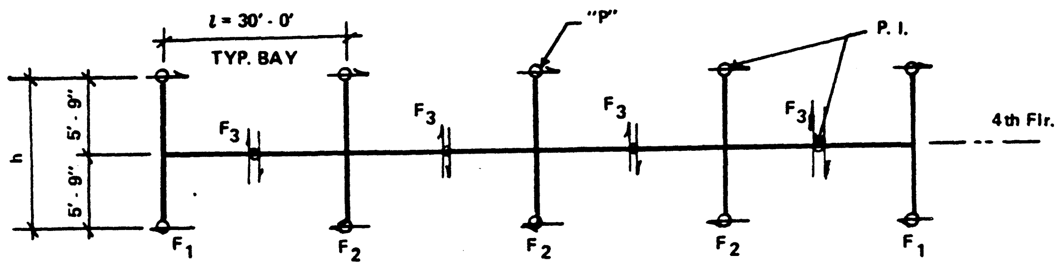


Fig. 3.3. Earthquake shears acting on frame

Preliminary Design of Frame:

Design will be limited to 4th floor girders and 3rd to 5th floor columns (one tier), with the Portal Method being employed for preliminary design, assuming points of inflection at mid-length of members (see Fig. 3.3).

$$2F_1 + 3F_2 = V_{A,3} = V_{D,3}$$

Since drift usually controls the design of moment frames, story shear will be that specified for drift calculations.

$$V_{A,3} = V_{D,3} = 0.51V_3 = (0.51)(216) = 110 \text{ kips}$$

$$2F_1 + 3F_2 = 110$$

$$\text{Since } F_1 = F_2/2, \quad 4F_2 = 110$$

$$F_2 = 27.5 \text{ kips}$$

Summation of moments about point **P** (see Fig. 3.3):

$$30.0F_2 = 11.50F_3 = 11.50\ (27.5)$$

$$F_3 = 10.5 \text{ kips}$$

The story drift can be determined by the following relationships:

$$\Delta_s = \Delta_c + \Delta_g$$

$$\Delta_c = \frac{Fh^3}{12EI_c}; \quad \Delta_g = \frac{Flh^2}{12EI_g}$$

where

F = Column shear
h = Story height
I_c = Moment of inertia of column
I_g = Moment of inertia of girder
l = Girder length
Δ_s = Story drift
Δ_c = Contribution of column to drift
Δ_g = Contribution of girder to drift

Allowable story drift is limited to $0.005h$:

$$\Delta_s = 0.005h = (0.005)(12 \times 11.5)$$

$$= 0.69 \text{ in.}$$

$$0.69 = \frac{Fh^3}{12EI_c} + \frac{Flh^2}{12EI_g}$$

There are many possible column and girder combinations that could satisfy this drift criteria. A preliminary column size can be determined by computing the axial load and moment on an interior column of a third story rigid bent.*

$$P_T = P_E + P_V$$

where

P_E = Axial force due to earthquake load
P_V = Axial force due to vertical load
P_T = Axial force due to total load

$P_E \approx 0$ for an interior column

$$P_V = \text{Roof} + 4 \text{ Floors} + \text{Curtain Wall}$$
$$= (30.0 \times 13.75)(0.067 + 0^{**}) + 4(30.0 \times 13.75) \times (0.085 + 0.020^{\dagger}) + (30.0 \times 60.5)(0.015)$$
$$= 28 + 173 + 27 = 228 \text{ kips}$$

$$P_T = 0 + 228 = 228 \text{ kips (at third story)}$$

Taking moments about x-x axis of column:

$$M_T = M_E + M_V$$

where

M_E = Moment due to earthquake load
M_V = Moment due to vertical load
M_T = Moment due to total load

$$M_E = (K^*)\left(\frac{h}{2} \times F_2\right) = (0.67)\left(\frac{11.5}{2} \times 27.5\right)$$
$$= 106 \text{ kip-ft}$$

$$M_V \approx 0$$

$$M_T = 106 + 0 = 106 \text{ kip-ft}$$

Converting this moment into an equivalent axial load, P_{equiv}, to obtain an approximate column size:

$$P_{equiv} \approx P + M_xB_x + M_yB_y$$ (pg. 3-8, AISC Manual[11])

$$M_yB_y \approx 0$$

**Reactions obtained in the drift analysis must be multiplied by K, the seismic coefficient, in order to obtain corresponding reactions for use in a stress analysis.*

***Roof live load not required in seismic design.*

†Floor live load reduced (UBC Sect. 2306).

$$P_{equiv} = \frac{228 + (106 \times 12)(0.185)}{1.33} = 348 \text{ kips}$$

(The factor of 1.33 reduces the required load to account for $\frac{1}{3}$ increase in allowable stress, UBC Sect. 2303.)

Select: W14×74 column (P_{allow} = 384 kips)

For a W14×74 column:

$$0.69 = \frac{Fh^3}{12EI_c} + \frac{Flh^2}{12EI_g}$$
$$= \frac{(27.5)(11.5)^3(1728)}{(12)(29{,}000)(797)} + \frac{(27.5)(30.0)(11.5)^2(1728)}{(12)(29{,}000)I_g}$$

Solving for I_g:

$$0.69 = 0.26 + 542/I_g$$
$$I_g = 1260 \text{ in.}^4$$

Select: W24×55 girder (I_g = 1340 in.^{4})

Stress Check of Frame:

W24×55 girder:

$$M_T = M_E + M_V$$
$$M_E = (F_3)(l/2)(K) = (10.5)(30.0/2)(0.67) = 106 \text{ kip-ft}$$
$$M_V \approx \frac{wl^2}{12} \text{ (where } w = \text{ uniform load on member)}$$
$$= (0.96 + 0.17)(30.0)^2/12 = 85 \text{ kip-ft}$$
$$M_T = 106 + 85 = 191 \text{ kip-ft}$$
$$S_{req} = \frac{M_T}{1.33F_b} = \frac{(191)(12)}{(1.33)(24.0)} = 71.8 \text{ in.}^3$$

W24×55 (S = 114 in.^{3}) **o.k.**

W14×74 column:

$$P_T = 228 \text{ kips}$$
$$M_T = 106 \text{ kip-ft (about } x\text{-axis)}$$

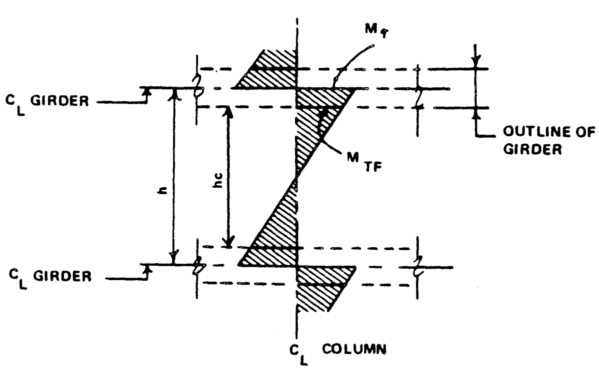


Figure 3.4

Most shear connections of a girder to a column web will result in a very small moment in the column. Thus, the column moment about the y-axis is small and will be neglected.

Reducing column moment from the center line to the face of the girder (see Fig. 3.4):

$$M_{TF} = M_T(h_c/h)$$

where

h_c = Clear height of column

M_{TF} = Moment due to total load, reduced to the face of girder

$$M_{TF} = (106)\left(\frac{11.5 - 2.0}{11.5}\right) = 87.4 \text{ kip-ft}$$

K_y = 1.0, since braced frame provides column stability

K_x > 1.0, since column stability depends upon the bending stiffness of the frame itself. Use Fig. C1.8.2, pg. 5-139 of the AISC Manual, to determine K_x.

$$G_A = G_B = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_G}{L_G}} = \frac{2\left(\frac{797}{11.5}\right)}{2\left(\frac{1340}{30.0}\right)} = 1.55$$

Therefore, K_x = 1.5

$$\left(\frac{Kl}{r}\right)_y = \left(\frac{1.0 \times 11.5 \times 12}{2.48}\right) = 55.7; \quad F_a = 17.8 \text{ ksi}$$

$$\left(\frac{Kl}{r}\right)_x = \left(\frac{1.5 \times 11.5 \times 12}{6.05}\right) = 34.2; \quad F_a = 19.6 \text{ ksi}$$

$$F'_{ex} = 129 \text{ ksi}$$
$$C_{mx} = 0.85$$
$$f_a = 228/21.8 = 10.5 \text{ ksi}$$
$$f_{bx} = (87.4 \times 12)/112 = 9.3 \text{ ksi}$$
$$F_{bx} = 0.66F_y = 24.0 \text{ ksi}$$

Stress level must satisfy the following ratio:

$$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right)F_{bx}} + \frac{C_{my}f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right)F_{by}} \leqslant 1.33$$

$$\frac{10.5}{17.8} + \frac{(0.85)(9.3)}{\left(1 - \frac{10.5}{129 \times 1.33}\right)24.0} + 0$$
$$= 0.59 + 0.35 = 0.94 \leqslant 1.33$$

W14×74 column **o.k.**

The preceding design meets the minimum requirements of the 1976 UBC. However, it should not be inferred from this example that a seismic design to exact code minimums is necessarily a recommended design.

Note that if a more accurate check is desired, a computer frame program can be used for determining both the drift and stress level of the frame.

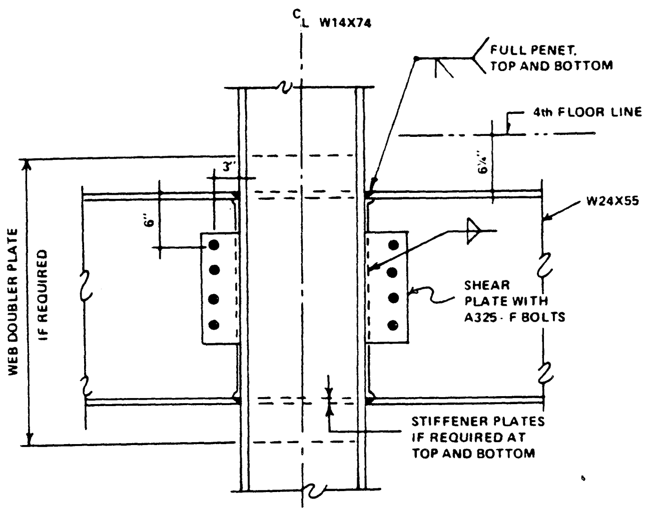


Fig. 3.5. Connection detail

Connection Design, Including Panel Zone (see Fig. 3.5):

Girder Moment Connection:

Section 2722(d) of the UBC requires that, "Each beam or girder moment connection to a column shall be capable of developing in the beam the full plastic capacity of the beam or girder."

Research work performed at the University of California at Berkeley and Lehigh University has demonstrated that the full plastic moment capacity of the girder can be developed by welding only the flanges.[12] Therefore, a full penetration weld will be employed for connecting the girder flanges to the column along with a single shear plate connection at the web. This is the most economical way of achieving a full moment connection according to a connection cost study conducted in 1973.[13]

Girder Shear Connection:

To provide an adequate shear connection of the girder to the column, it will be necessary to investigate two loading conditions: Code loads per Chapter 23 of the UBC (Working Stress Design), and developing the full plastic capacity of the girder. The larger of the two results will determine the required shear connection.

For code loads:

$$F_T = F_E + F_V$$

where

F_E = Shear due to earthquake load
F_V = Shear due to vertical load
F_T = Shear due to total load

$$F_T = (K)(F_3) + (wl/2)$$
$$= (0.67)(10.5) + (0.96 + 0.17)(30.0/2)$$
$$= 7.0 + 17.0 = 24.0 \text{ kips}$$

Using 7/8-in. ϕ A325-F high-strength bolts:

$$n = \frac{24.0}{9.02 \times 1.33} = 2.0 \text{ bolts}$$

For full plastic moment capacity of girder:

$$F_T = F_E + F_V$$

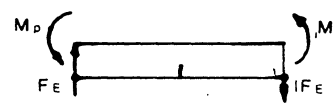


Figure 3.6

$$F_E = 2M_p/l \quad \text{(see Fig. 3.6)}$$

$F_V = 1.3(wl/2)$ (where 1.3 is load factor for plastic design; see Sect. 2.1 of AISC Specification)

$$F_T = (2)(402)/30.0 + (1.3)(17.0) = 48.9 \text{ kips}$$

Using 7/8-in. ϕ A325-F bolts and increasing their capacity by a factor of 1.7 as permitted by Sect. 2.8 of the AISC Specification:

$$n = \frac{48.9}{9.02 \times 1.7} = 3.2 \text{ bolts (governs)}$$

Use: Four 7/8-in. diam. A325-F high-strength bolts

Shear plate:

Based on the shear force resulting from the plastic moment capacity of the girder controlling the design, along with a shear plate eccentricity of 3 in.:

$$M_T = (F_T)(3 \text{ in.}) = (48.9)(3) = 147 \text{ kip-in.}$$

Try shear plate 3/8-in. thick × 1 ft-0 in. long:

$$f_v = \frac{(1.5)(48.9)}{(12.0 \times 0.375)} = 16.3 \text{ ksi}$$

$F_v = 0.55F_y = 19.8$ ksi > 16.3 ksi **o.k.**

$$f_b = \pm\frac{M_T}{Z} = \frac{147}{(0.375)(12)^2/4} = 10.9 \text{ ksi}$$

$F_b = F_y = 36.0$ ksi > 10.9 ksi **o.k.**

Use: Shear plate 4½ × 3/8 × 1 ft-0 in.

Using two-sided fillet weld to column:

Force per linear inch of weld:

$$f_V = \frac{48.9}{(2)(12)} = 2.04 \text{ kips/in. (vert. force)}$$

$$f_H = \frac{147}{(2)(12)^2/4} = 2.04 \text{ kips/in. (horiz. force)}$$

$$f_R = \sqrt{(2.04)^2 + (2.04)^2}$$
$$= 2.90 \text{ kips/in. (resultant force)}$$

Using E70 electrodes, the plastic design capacity of a $\frac{1}{16}$-in. fillet weld is:

$$f_{cap} = (1.7 \times 21.0)(0.707 \times \tfrac{1}{16})(1.00)$$
$$= 1.58 \text{ kips/in.}$$

$$n_{req'd} = 2.90/1.58 = 1.83 \text{ (or } \tfrac{1}{8}\text{-in. fillet weld)}$$

Use: $\frac{1}{4}$-in. fillet welds (minimum required by AISC Specification, Table 1.17.5)

Column stiffener plates:

Requirements for stiffeners will be determined on the basis of Sect. 1.15.5 of the AISC Specification.

Opposite compression flange, required if:

$$t < \frac{C_1 A_f}{t_b + 5k} \text{ or } t \leqslant \frac{d_c\sqrt{F_y}}{180}$$

$$\frac{C_1 A_f}{t_b + 5k} = \frac{(1.0)(7.00 \times 0.50)}{0.50 + 5(1.5)} = 0.44 \text{ in.}$$

$$\frac{d_c\sqrt{F_y}}{180} = \frac{(11.25)(\sqrt{36.0})}{180} = 0.37 \text{ in.}$$

$t = 0.45$ in. $>$ 0.44 in. and 0.37 in.

$\therefore$ **stiffeners not required.**

Opposite tension flange, required if:

$$t_f < 0.4\sqrt{C_1 A_f}$$

$$0.4\sqrt{C_1 A_f} = 0.4\sqrt{(1.0)(7.00 \times 0.50)} = 0.75 \text{ in.}$$

$t_f = 0.78$ in. $>$ 0.75 in.

$\therefore$ **stiffeners not required.**

Column panel zone:

Due to earthquake forces, the moments at each end of the girder will act in the same direction concurrently. This can result in large shears in the column panel zone, especially at interior columns of moment frames.

The vertical load moments, M_V, tend to cancel each other. However, the earthquake load moments, M_E, are additive, as shown in Fig. 3.7.

$$F \approx \frac{2M_E}{0.95d_g}$$

where

F = Axial force at the girder flanges; when coupled, resulting moments are equivalent to the girder earthquake moments

d_g = Depth of girder

M_E = 106 kip-ft

$$F = \frac{2(106 \times 12)}{(0.95)(23.6)} = 113 \text{ kips}$$

Taking moments about point **P** (Fig. 3.7):

$$F_2 = \frac{2M_E}{h} = \frac{2(106 \times 12)}{(11.5 \times 12)} = 18 \text{ kips}$$

F_1 = Panel zone shear $= F - F_2 = 95$ kips

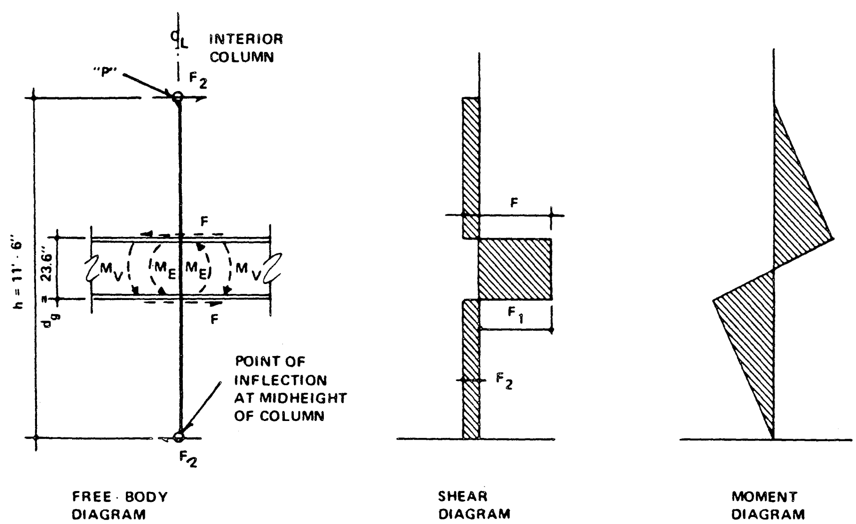


Figure 3.7

Computing shear stress in panel zone:

$$f_v = \frac{F_1}{A_{web}} = \frac{95}{(14.19 \times 0.45)} = 14.9 \text{ ksi}$$

$$F_v = 14.5 \times 1.33 = 19.3 \text{ ksi} > 14.9 \text{ ksi} \quad \textbf{o.k.}$$

$\therefore$ **web doublers not required at panel zone**

If web doubler plates had been required, the details shown in Fig. 3.8 are recommended.

Column splice details:

Partial penetration welds are usually adequate for resisting reactions at the column splice. For connection details see pgs. 8 and 9 of Ref. 13.

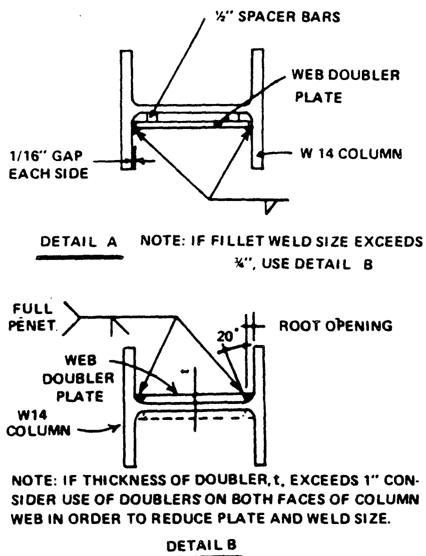


Fig. 3.8. Recommended web doubler plate details

Braced Frame Design

North-South Seismic Forces:

$$V = ZIKCSW \qquad (14\text{-}1)$$

$Z = 1.00$ for Zone No. 4
$I = 1.00$ per UBC Table No. 23-K
$K = 1.00$ per UBC Table No. 23-I
(framing system has a complete "vertical load-carrying space frame")

$$V = (1.00)(1.00)(1.00)(CSW) = CSW$$

$$T = \frac{0.05h_n}{\sqrt{D}} = \frac{(0.05)(83.0)}{\sqrt{75.0}} = 0.48 \text{ sec.} \qquad (14\text{-}3A)$$

Formula (14-3A) of the UBC provides a reasonable estimate for the period of a braced frame. (Once the member sizes of the braced frame are determined, then the period of the building can be accurately determined by a properly substantiated analysis.)

$$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{0.48}} = 0.096 \qquad (14\text{-}2)$$

$$\frac{T}{T_s} = \frac{0.48}{1.00} = 0.48 \leq 1.00$$

$$S = 1.0 + \frac{T}{T_s} - 0.5\left(\frac{T}{T_s}\right)^2 \qquad (14\text{-}4)$$

$$= 1.0 + 0.48 - 0.5(0.48)^2 = 1.36$$

$$CS = (0.096)(1.36) = 0.131 \leq 0.14 \quad \textbf{o.k.}$$

$$W = 5{,}930 \text{ kips}$$

$$V = CSW = (0.096)(1.36)(5{,}930)$$
$$= 775 \text{ kips (total lateral force)}$$

Table 3-2. Distribution of Earthquake Forces and Story Shears

Floor Level	h_x (ft)	w_x (kips)	$w_x h_x \times 10^{-2}$	$\frac{w_x h_x}{\Sigma w_i h_i}$	F_x[a] (kips)	V_x[a] (kips)	F_x[b] (kips)	V_x[b] (kips)
R	83.0	687	570	0.203	158	—	198	—
7	71.5	874	625	0.222	172	158	215	198
6	60.0	874	524	0.187	145	330	181	413
5	48.5	874	424	0.151	117	475	146	594
4	37.0	874	324	0.115	89	592	111	740
3	25.5	874	222	0.079	61	681	76	851
2	14.0	874	122	0.043	33	742	41	927
1	—	—	—	—	—	775	—	968
Σ	—	—	2811	1.000	775	—	968	—

[a] Forces and shears to determine drift of frames and overturning at base of building.
[b] Forces and shears for drift and overturning multiplied by 1.25 to obtain corresponding values for determining frame member sizes and connections. See Sect. 2314(j)1G of 1976 UBC (or Sect. 1(J)1g of 1974 SEAOC Code, Appendix A of this paper).

Since $T \leq 0.70$, $F_t = 0$

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} = \frac{(775)w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

See Table 3-2 for distribution of lateral forces over the height of the building.

Distribution of Earthquake Forces:

As determined previously in the moment frame design,

$$e = (0.05)(120) = 6.0 \text{ ft}$$
$$R_1 = R_5 = 4.00 \text{ (approx.)}$$
$$R_A = R_D = 1.00 \text{ (approx.)}$$

Shear distribution in N-S direction:

$$V_{1,x} = (R_1)\left[\frac{V_x}{\Sigma R_{N\text{-}S}} \pm \frac{(V_x e)(d)}{\Sigma R_y d^2}\right] = V_{5,x}$$

$$\Sigma R_{N\text{-}S} = 2(4.00) = 8.00$$

$$\Sigma R_y d^2 = 2(1.00)(37.5)^2 + 2(4.00)(60.0)^2 = 31{,}600$$

$$V_{1,x} = (4.00)\left[\frac{V_x}{8.00} \pm \frac{(V_x \times 6.00)(60.00)}{31{,}600}\right]$$
$$= (4.00)[0.125V_x + 0.011V_x]$$
$$= 0.545V_x = V_{5,x}$$

Bracing Systems:

Possible bracing systems that might be utilized are shown in Fig. 3.9.

An important design consideration in selecting a bracing system is overturning due to earthquake forces. Overturning moments are as shown in Table 3-3.

Table 3.3. Earthquake Overturning Moments

Floor Level	V_x (kips)	Story Height $= h_x$ (ft)	$V_x h_x$	Moment: $M_x = V_x h_x$[a] (kip-ft)	Moment: $1.25(M_x)$[b] (kip-ft)
7	158	11.5	1820	1820	2280
6	330	11.5	3800	5620	7020
5	475	11.5	5460	11,080	13,800
4	592	11.5	6800	17,880	22,300
3	681	11.5	7830	25,710	32,200
2	742	11.5	8530	34,240	42,800
1	775	14.0	10,860	45,100	56,400
Σ	—	—	45,100	45,100	56,400

[a] Moment to determine overturning at base of building.
[b] Moment to determine frame member sizes and connections per Sect. 2314(j)1G of 1976 UBC (or Sect. 1(J)1g of 1974 SEAOC Code, Appendix A of this paper).

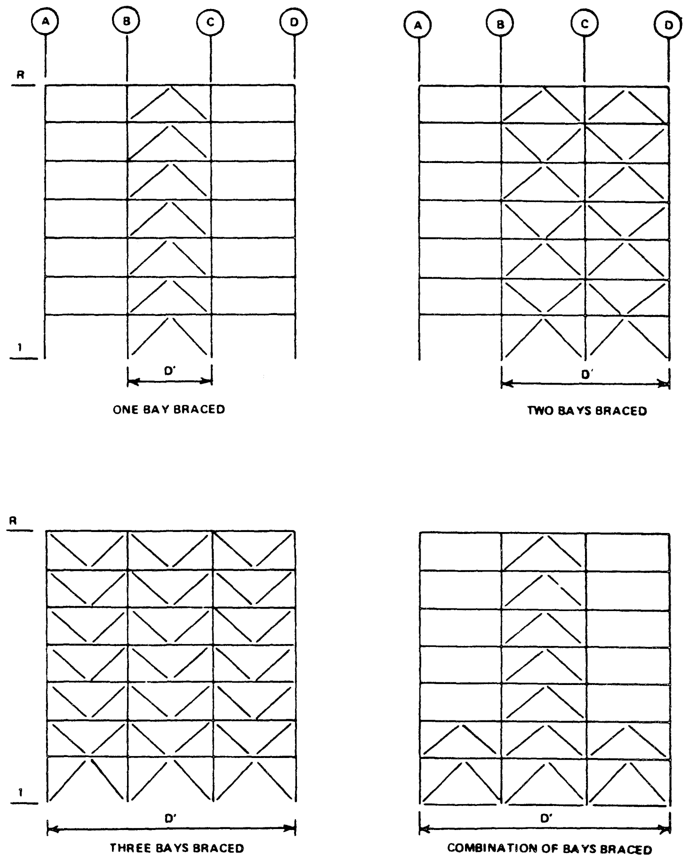


Fig. 3.9. Bracing systems

Overturning moment is distributed to the frames in the same proportion as the shears:

$$M_{1,x} = M_{5,x} = 0.545M_x$$

where

M_x = Total earthquake moment on building at story x

$M_{y,x}$ = Earthquake moment on a braced frame, referred to that frame on column line y at level x

$$M_{1,1} = M_{5,1} = (0.545)(45{,}100)$$
$$= 24{,}600 \text{ kip-ft(at base)}$$

This overturning moment must be resisted by the dead load of the braced portion of the frame. Consider the following cases at the base of the frames.

One Bay Braced:

$$M_{1,1} = M_{5,1} = 24{,}600 \text{ kip-ft}$$

Dead load of columns on lines **B** and **C**:

Roof	= (407)(0.067) =	27 kips
6 Floors	= 6(407)(0.085) =	208
Curtain Wall	= (1800)(0.015) =	27
Footing	≃	30
	$P_B = P_C$ =	292 kips

$$W_{DL} = 2(292) = 584 \text{ kips}$$

$$M_R = W_{DL}(D'/2)$$

where

D' = Width of a braced frame at base

M_R = Dead load resisting moment of a frame

W_{DL} = Dead load of a braced frame

$$M_R = (584)(25/2) = 7{,}300 < 24{,}600 \text{ kip-ft} \quad \textbf{n.g.}$$

Overturning exceeds resisting moment. This indicates that the frame is unstable unless the resisting moment is increased by using caissons, piles, or other means which will increase the dead load of the braced portion of the frame.

Two Bays Braced:

By comparison with one bay braced, the frame would be unstable unless caissons, etc. are used.

Three Bays Braced:

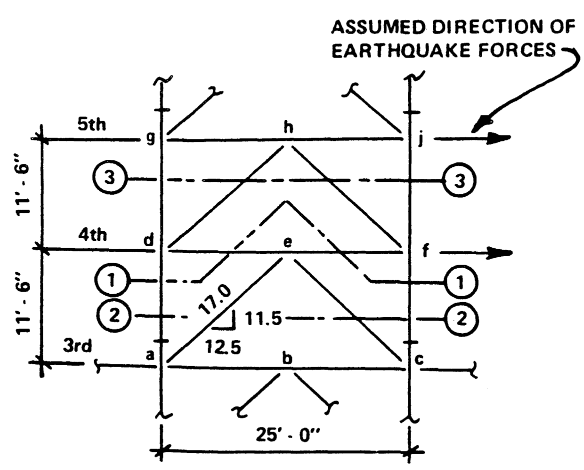
$$M_{1,1} = M_{5,1} = 24{,}600 \text{ kip-ft}$$

Dead load of columns on lines **A**, **B**, **C**, and **D**:

$$W_{DL} = 2(292) + 2(191) = 966 \text{ kips}$$

$$M_R = (966)(75.0/2) = 36{,}200 > 24{,}600 \text{ kip-ft} \quad \textbf{o.k.}$$

This frame is stable without utilizing caissons; however, to reduce the number of braced frame members and connections, the "Combination of Bays Braced" framing system is used (see Fig. 3.9). This system will spread the overturning out to the base in the same way as the "Three Bays Braced" system, but more efficiently.



PARTIAL ELEVATION

Figure 3.10

Design of Braced Frame Members:

Design of the frame will be limited to 4th floor girders, 3rd to 5th floor columns (one tier), and 4th story braces.

Forces Due to Earthquake Loading (see Fig. 3.10):

Since $V_{1,x} = V_{5,x} = 0.545V_x$, the shears at the 3rd and 4th stories are:

$$V_{1,4} = V_{5,4} = (0.545)(740) = 403 \text{ kips}$$

$$V_{1,3} = V_{5,3} = (0.545)(851) = 464 \text{ kips}$$

Overturning moment at the 4th floor is:

$$M_{1,4} = M_{5,4} = (0.545)(22{,}300) = 12{,}200 \text{ kip-ft}$$

At Section 1-1:

Taking moments about point **f** and solving for the axial force in member **ad**:

$$\Sigma M_f = 0 = 12{,}200 - (25.0)(P_{ad})$$

$$P_{ad} = 12{,}200/25.0 = 487 \text{ kips}$$

$$P_{cf} = -487 \text{ kips}$$

At Joint **e**:

$$\Sigma F_y = 0 = -\left(\frac{11.5}{17.0}\right)(P_{ae}) - \left(\frac{11.5}{17.0}\right)(P_{ce})$$

$$P_{ae} = -P_{ce}$$

By taking $\Sigma F_x = 0$, it can be shown that $P_{de} = -P_{ef}$.

At Section 2-2:

$$\Sigma F_x = 0 = 464 - \left(\frac{12.5}{17.0}\right)(P_{ae}) + \left(\frac{12.5}{17.0}\right)(P_{ce})$$

Since $P_{ae} = -P_{ce}$,

$$464 = 2\left(\frac{12.5}{17.0}\right)(P_{ae})$$

$$P_{ae} = 315 \text{ kips}$$

$$P_{ce} = -315 \text{ kips}$$

At Section 3-3:

$$\Sigma F_x = 0 = 403 - \left(\frac{12.5}{17.0}\right)(P_{dh}) + \left(\frac{12.5}{17.0}\right)(P_{fh})$$

$$P_{dh} = 274 \text{ kips}$$

$$P_{fh} = -274 \text{ kips}$$

At Section 1-1:

$$\Sigma F_x = 0 = 464 + P_{de} - P_{ef},$$

Since $P_{de} = -P_{ef}$,

$$P_{de} = -232 \text{ kips}$$

$$P_{ef} = 232 \text{ kips}$$

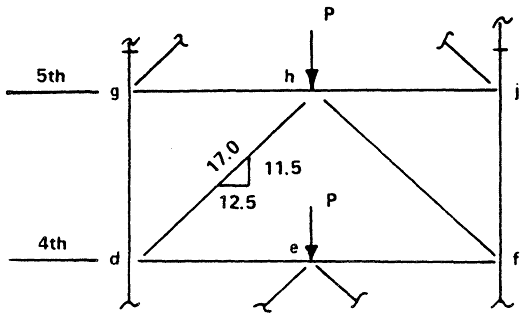


Figure 3.11

Forces Due to Vertical Loading at Brace Intersection (see Fig. 3.11):

$$P = \text{Floor} + \text{Curtain Wall}$$
$$= (16.25 \times 12.5)(0.082 + 0.048) + (11.5 \times 12.5)(0.015)$$
$$= 28.6 \text{ kips}$$

At Joint **h**:

$$\Sigma F_y = 0 = -28.6 - \left(\frac{11.5}{17.0}\right)(P_{dh}) - \left(\frac{11.5}{17.0}\right)(P_{fh})$$

From $\Sigma F_x = 0$ along Sect. 1-1 and then Joints **d** and **f**, it can be shown that $P_{dh} = P_{fh}$. Thus,

$$28.6 = -2\left(\frac{11.5}{17.0}\right)(P_{dh})$$

$$P_{dh} = -21.1 \text{ kips}; \qquad P_{fh} = -21.1 \text{ kips}$$

At Joint **d**:

$$\Sigma F_x = 0 = \left(\frac{12.5}{17.0}\right)(P_{dh}) + P_{de}$$

$$P_{de} = -\left(\frac{12.5}{17.0}\right)(-21.1) = 15.5 \text{ kips}$$

Design of Brace (4th Story):

$$P_T = P_E + P_V$$

where

P_E = Axial force due to earthquake load
P_V = Axial force due to vertical load
P_T = Axial force due to total load

$$P_E = \pm 274 \text{ kips}$$

$$P_V = -21 \text{ kips}$$

$$P_T = -274 - 21 = -295 \text{ kips}$$

$$\text{or } P_T = +274 + 0 = +274 \text{ kips (neglecting vertical load)}$$

Using theoretical length of brace, rather than the actual (somewhat smaller) length:

$$(Kl)_y = (1.0)(17.0) = 17.0 \text{ ft}$$

Taking one-third increase on brace capacity:

$$P_{equiv} = -295/1.33 = -222 \text{ kips}$$

$$\text{or } P_{equiv} = +274/1.33 = +206 \text{ kips}$$

Try W14×61; $P_{allow} = 268$ kips (C):

Check tension capacity:

If four $1\frac{3}{16}$-in. diam. holes occur in flanges at a given cross section,

$$A_{net} = 17.9 - 4(1.25 \times 0.64) = 14.7 \text{ sq. in. (governs)}$$

$$A_{net(max)} = (0.85)(17.9) = 15.2 \text{ sq. in.}$$

$$P_{allow} = (14.7)(22.0) = 324 \text{ kips (T)} > +206 \text{ kips} \quad \textbf{o.k.}$$

Use: W14×61 brace

Design of Columns (3rd to 5th Floor):

$$P_T = P_E + P_V$$

Using loads at 3rd story:

$$P_E = \pm 487 \text{ kips}$$

$$P_V = \text{Roof} + 4 \text{ Floors} + \text{Curtain Wall}$$
$$= (25.0 \times 16.25)(0.067 + 0) + 4(25.0 \times 16.25)(0.085 + 0.020) + (25.0 \times 60.5)(0.015)$$
$$= 27 + 171 + 23$$
$$= 221 \text{ kips (C)} = -221 \text{ kips}$$

$$P_T = -487 - 221 = -708 \text{ kips (C)}$$

$$P_{equiv} = -708/1.33 = -532 \text{ kips}$$

$$(Kl)_y = (Kl)_x = (1.0)(11.5) = 11.5 \text{ ft}$$

As shown in the AISC Manual, Table C1.8.1, pg. 5-138, a value of $K = 1.0$ is very conservative for a continuous column.

Use: W14×95 column ($P_{allow} = 538$ kips)

Design of Girder (4th Floor):

$$P_T = P_E + P_V$$

$$P_E = \pm 232 \text{ kips}$$

$$P_V = +16 \text{ kips}$$

$$P_T = -232 + 0 = -232 \text{ kips (neglecting vertical load)}$$

$$\text{or } P_T = +232 + 16 = +248 \text{ kips}$$

$$P_{equiv} = -232/1.33 = -174 \text{ kips}$$

$$\text{or } P_{equiv} = +248/1.33 = +187 \text{ kips}$$

Tributary floor loads will cause bending of the girder about its weak axis. The girder is continuous over two spans.

$$M_V = wl^2/8$$

$$\begin{aligned} w &= \text{Floor} + \text{Curtain Wall} + \text{Girder} \\ &= (3.25)(0.072 + 0.050) + (11.5)(0.015) + 0.070 \\ &= 0.63 \text{ kips/ft} \end{aligned}$$

$$M_V \simeq (0.63)(12.5)^2/8 = 12.3 \text{ kip-ft}$$

Try W14×61:

$$\left(\frac{Kl}{r}\right)_y = \frac{(1.0)(12.5 \times 12)}{2.45} = 61$$

$$F_a = 17.3 \text{ ksi}; \qquad F'_{ey} = 40.1 \text{ ksi}$$

$$f_a = \frac{232}{17.9} = 13.0 \text{ ksi}$$

$$f_{by} = \frac{12.3 \times 12}{21.5} = 6.9 \text{ ksi}$$

Taking $C_{my} = 1.0$,

$$\frac{f_a}{F_a} + \frac{C_m f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leqslant 1.33$$

$$\frac{13.0}{17.3} + \frac{(1.0)(6.9)}{\left(1 - \frac{13.0}{40.1}\right) 27.0} = 0.75 + 0.38$$

$$= 1.13 \leqslant 1.33 \quad \textbf{o.k.}$$

Use: W14×61 girder

Approximate Drift and Building Period:

The drift at the top of the frame due to earthquake forces can by approximated by using the following equation:

$$\Delta = \Delta_{ch} + \Delta_{web}$$

where

Δ = Drift at top of frame

Δ_{ch} = Drift due to chord members (see Fig. 6.5, Sect. 6)

$$= \frac{4F_t H^3}{3EAD^2} + \frac{4(V - F_t)H^3}{5.5EAD^2}$$

Δ_{web} = Drift due to web members $= \Sigma\Delta_{horiz} + \Sigma\Delta_{diag}$

Δ_{horiz} = Drift due to stress in horizontal web members (see Fig. 3.12)

Δ_{diag} = Drift due to stress in diagonal web members (see Fig. 3.13)

Therefore,

$$\Delta_{web} = \Sigma\Delta_{horiz} + \Sigma\Delta_{diag} = \frac{nf_1l_1}{2E} + \frac{nf_2l_2}{E \cos\theta}$$

where

f_1 = Average stress in horizontal members due to earthquake load

f_2 = Average stress in diagonal members due to earthquake load

n = Number of stories

Due to the large offset in the bracing system at the third floor, the drift due to the chord members can be approximated by using only the narrow portion of the bracing system above the third floor.

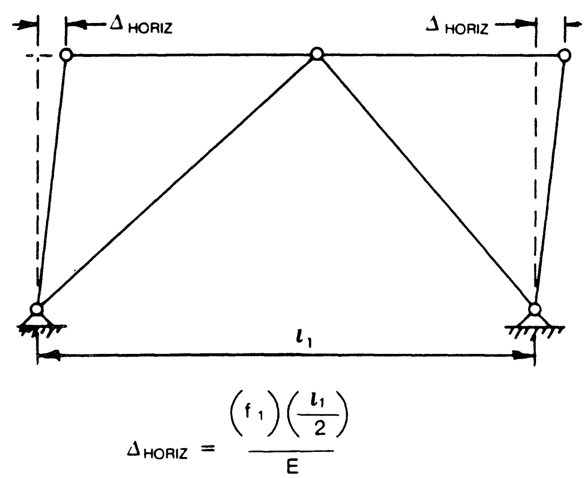


Fig. 3.12. Drift due to stress in horizontal web members

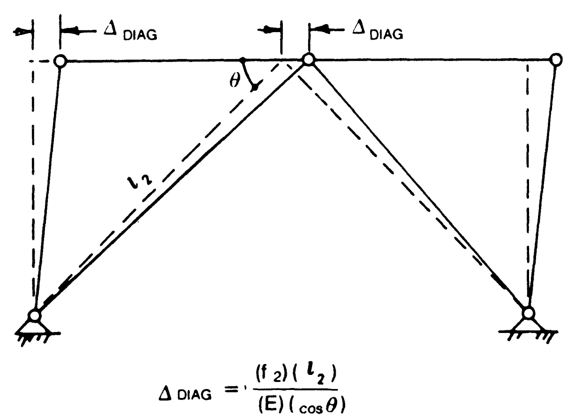


Fig. 3.13. Drift due to stress in diagonal web members

$$\Delta_{ch} = \frac{4F_tH^3}{3EAD^2} + \frac{4(V - F_t)H^3}{5.5EAD^2}$$

$F_t = 0$

$V = 0.545V_x = 0.545V_{1,3} = (0.545)(681)^*$
$= 372$ kips

$A = 2 \times 21.8 = 43.6$ sq.in.
(Assuming average column is W14×74 from 3rd floor to roof.)

$D = 25.0$ ft

$H = 5(11.5) = 57.5$ ft

$$\Delta_{ch} = 0 + \frac{4(372)(57.5)^3}{5.5(29{,}000)(43.6)(25.0)^2}$$
$$= 0.065 \text{ ft} = 0.78 \text{ in.}$$

$$\Delta_{web} = \frac{nf_1l_1}{2E} + \frac{nf_2l_2}{E \cos \theta}$$

$^*f_1 = \text{girder stress} \simeq P_E/1.25A$
$= 232/(1.25 \times 17.9) = 10$ ksi (4th floor)

$^*f_2 = \text{bracing stress} \simeq P_E/1.25A$
$= 274/(1.25 \times 17.9) = 12$ ksi (4th story)

$$\Delta_{web} = \frac{(7)(10)(25.0)}{(2)(29{,}000)} + \frac{(7)(12)(17.0)}{(29{,}000)(12.5/17.0)}$$
$$= 0.097 \text{ ft} = 1.16 \text{ in.}$$

Therefore,

$$\Delta = \Delta_{ch} + \Delta_{web}$$
$$= 0.78 + 1.16 = 1.94 \text{ in.}$$

Allowable building drift is limited to $0.005H$:

$$\Delta_{allow} = (0.005)(83.0)$$
$$= 0.415 \text{ ft} = 4.98 \text{ in.} > 1.94 \text{ in.} \qquad \textbf{o.k.}$$

The period of the building in the north-south direction can be approximated by the equation

$$T = 0.25\sqrt{\Delta/C_1}$$

where

$\Delta = 1.94$ in.

$C_1 = ZICS = 0.131$

$T = 0.25\sqrt{1.94/0.131} = 0.96$ sec.

Based on this period,

$$C = \frac{1}{15\sqrt{T}} = \frac{1}{15\sqrt{0.96}} = 0.068$$

**Stresses used in drift calculations should not include the 25% increase in force level used in determining member sizes and connections.*

Since $\frac{T}{T_s} = \frac{0.96}{1.00} = 0.96 \leqslant 1.00$,

$$S = 1.0 + \frac{T}{T_s} - 0.5\left(\frac{T}{T_s}\right)^2$$
$$= 1.0 + 0.96 - 0.5(0.96)^2 = 1.50 = S_{max}$$

$$CS = (0.068)(1.50) = 0.102 \leqslant 0.14 \qquad \textbf{o.k.}$$

Therefore,

$$V = ZIKCSW$$
$$= (1.00)(1.00)(1.00)(0.102)W = 0.102W$$

This indicates that the building in the north-south direction might be designed for a base shear of $0.102W$ rather than $0.131W$. The decision of whether or not to redesign would be left up to the judgement of the engineer, who should also consider that the particular bracing system shown is somewhat irregular; therefore, it might be interpreted that the code coefficients ($0.102W$) would not apply to this case.

Once the sizes of all the braced frame members are known, then the story and building drift can be better determined by using virtual work methods or a computer frame program. When the drift at each floor is known, then the building period can be accurately determined from Formula (14-3) of the UBC.

Connection Design:

Section 2314(j)1G of the UBC requires that braced frame connections shall be designed to develop the full capacity of the members or shall be based on the code forces without the ⅓ increase usually permitted for stresses resulting from earthquake forces. In the following design examples, the connections will be designed using the code forces specified without use of the ⅓ increase in allowable stresses.

Use 1⅛-in. ϕ A325-F bolts in single shear.

$F_{cap} = 14.9$ kips (AISC Manual, pg. 4-8)

Braces to Girder (4th Floor); see Fig. 3.14:

$P_{ae} = P_E + P_V = +315 - 21 = +294$ kips

$P_{ce} = -315 - 21 = -336$ kips

$P_{de} = -232 + 16 = -216$ kips

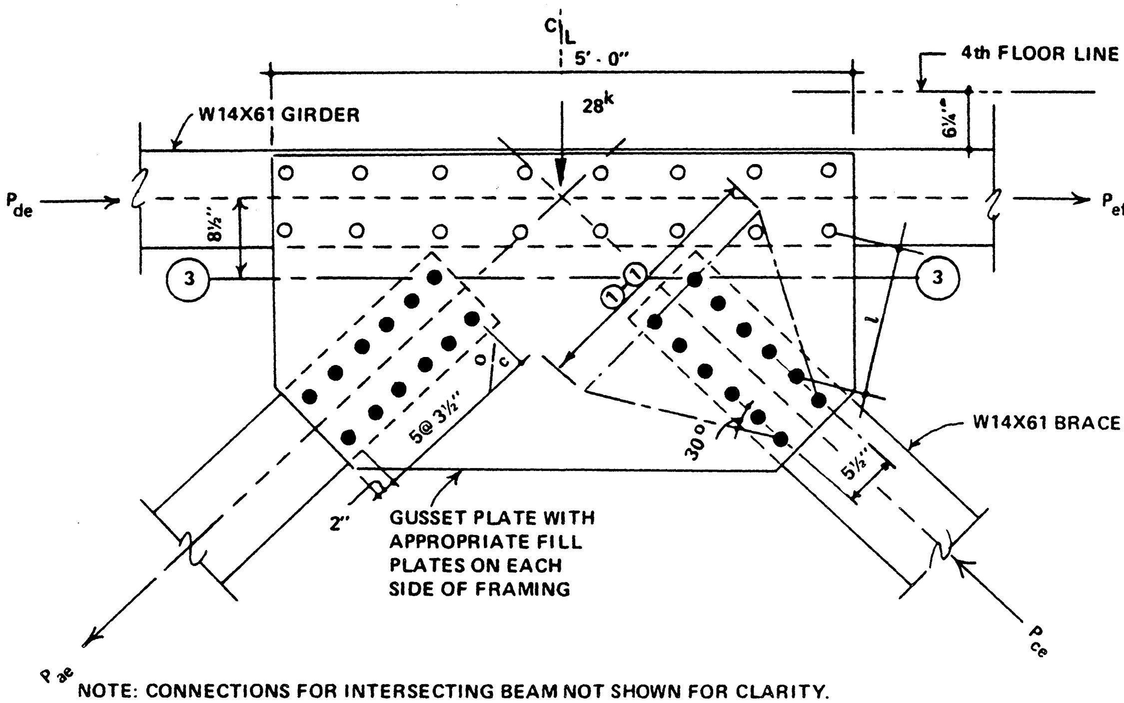
$P_{ef} = +232 + 16 = +248$ kips

Bolts to brace:

Using larger brace force,

$n_1 = 336/14.9 = 22.6$

Use: 24 1⅛-in. ϕ A325-F bolts



NOTE: CONNECTIONS FOR INTERSECTING BEAM NOT SHOWN FOR CLARITY.

AN ALTERNATE CONNECTION DETAIL WOULD BE TO FILLET WELD THE GUSSETS TO THE GIRDER RATHER THAN BOLTING.

Fig. 3.14. Brace to girder connection

Bolts to girder:

Assume 32 bolts:

$$f_H = (216 + 248)/32$$
$$= 14.5 \text{ kips/bolt (horizontal force)}$$

$$f_V = 28/32$$
$$= 0.9 \text{ kips/bolt (vertical force)}$$

$$f_R = \sqrt{(14.5)^2 + (0.9)^2}$$
$$= 14.5 \text{ kips/bolt (resultant force)}$$

Use: 32 1⅛-in. ϕ A325-F bolts

Gusset plate:

In order to facilitate the erection of the braced frame, 5⁄16-in. oversized holes will be used in the gusset plates. This will permit the steel erector greater latitude in plumbing the building. See UBC Standard Number 27-7.

The analysis of the gusset plates will be based on Whitmore's method[14] and the method of sections using the beam formulas.

Try ½-in. plate thickness:

At Section 1-1 (Fig. 3.14):

$$\text{Effective width} = 2(17.5 \times \tan 30°) + 5.5$$
$$= 25.7 \text{ in.}$$

$$f_a = \frac{336}{2(25.7 \times 0.50)} = 13.1 \text{ ksi}$$

$$F_a = 0.60F_y = 22.0 \text{ ksi} > 13.1 \text{ ksi} \qquad \textbf{o.k.}$$

At Section 2-2 (not shown):

Similar to Section 1-1 except located at end of tension brace. Deducting for two 17⁄16-in. diam. holes (use 1½-in. in computing net area, as per Supplement No. 2 to the AISC Specification, Sect. 1.14.5):

$$\text{Effective width} = 25.7 - 2(1.50) = 22.7 \text{ in.}$$

$$f_t = \frac{294}{2(22.7 \times 0.50)} = 12.9 \text{ ksi}$$

$$F_t = 0.60F_y = 22.0 \text{ ksi} > 12.9 \text{ ksi} \qquad \textbf{o.k.}$$

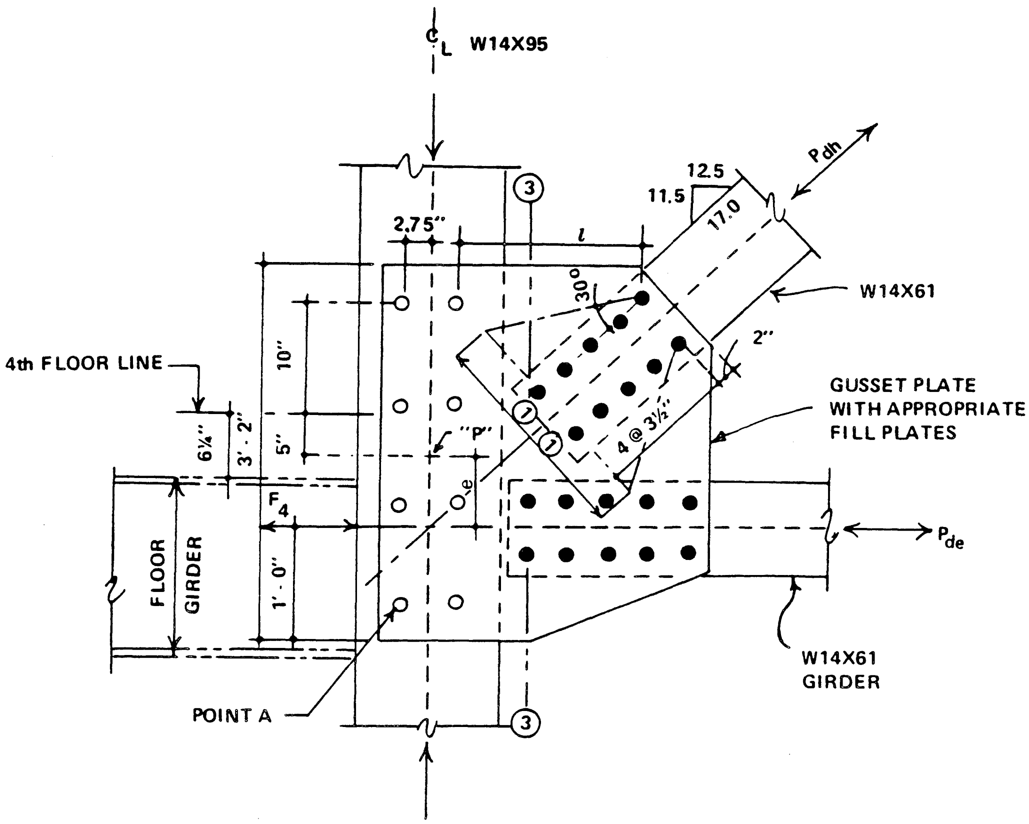
At Section 3-3 (Fig. 3.14):

$$f_v = \frac{1.5(216 + 248)}{2(60.0 \times 0.50)} = 11.6 \text{ ksi}$$

$$f_b = \pm \frac{(216 + 248)(8.5)}{2(0.50)(60)^2/6} = \pm 6.5 \text{ ksi}$$

$$f_a = \frac{-28}{2(60.0 \times 0.50)} = -0.5 \text{ ksi}$$

$$f_a \pm f_b = -0.5 \pm 6.5$$
$$= -7.0 \text{ ksi and } +6.0 \text{ ksi}$$



NOTE: CONNECTION FOR INTERSECTING GIRDER NOT SHOWN.

AN ALTERNATE CONNECTION DETAIL WOULD BE TO FILLET WELD THE GUSSETS TO THE COLUMN RATHER THAN BOLTING.

Fig. 3.15. Brace and girder to column connection

$F_v = 14.5 \text{ ksi} > 11.6 \text{ ksi}$ **o.k.**

F_a depends on Kl/r of plate between lateral supports.

$$\frac{Kl}{r} \simeq \frac{(1.0)(15)}{(0.29 \times 0.50)} = 103$$

$F_a = 12.6 > 7.0 \text{ ksi}$ **o.k.**

(The value of K can vary from 0.65 to 1.00· See Table C1.8.1, pg. 5-138, AISC Manual.)

Use: ½-in. plate thickness

Brace and Girder to Column (4th Floor); see Fig. 3.15:

$$P_{dh} = P_E + P_V = \pm 274 - 21$$
$$= -295 \text{ kips and } +253 \text{ kips}$$

(Forces are reversible; direction depends on direction of earthquake.)

$$P_{de} = P_E + P_V = \pm 232 + 16$$
$$= +248 \text{ kips and } -216 \text{ kips}$$

$$F_4 = \frac{V_{1,3} - V_{1,4}}{2} = \frac{464 - 403}{2} = 31 \text{ kips}$$

Force F_4 represents the earthquake load that is transferred through the fourth floor diaphragm to the floor girders on lines 1 and 5, then into *each* end of the braced frame. Assume that this force is transferred directly into the column through a connection not attached to the gusset plates.

Bolts to brace:

$$n_1 = 295/14.9 = 19.8$$

Use: 20 1⅛-in. ϕ A325-F bolts

Bolts to girder:

$$n_2 = 248/14.9 = 16.7$$

Use: 20 1⅛-in. ϕ A325-F bolts

Bolts to column:

$$F_V = \left(\frac{11.5}{17.0}\right)(P_{dh}) = \left(\frac{11.5}{17.0}\right)(295) = 200 \text{ kips}$$

$$F_H = \left(\frac{12.5}{17.0}\right)(P_{dh}) - P_{de} = F_4 = 31 \text{ kips}$$

where

F_H = Horizontal force acting on gusset plates
F_V = Vertical force acting on gusset plates

With the bolt group pattern shown, the horizontal force has an eccentricity of $e = (38/2) - 12 = 7$ in. with respect to the center of gravity of the bolt group at point **P**.

$n = 16$ bolts (total)

Polar moment of inertia about **P**:

$$J = 2[(2 \times 2)(15)^2 + (2 \times 2)(5)^2 + (2 \times 4)(2.75)^2] = 2{,}120 \text{ in.}^4$$

Maximum bolt force will be at point **A**. Thus,

$$f_{V_1} = F_V/n = 200/16 = 12.5 \text{ kips/bolt}$$

$$f_{H_1} = F_H/n = 31/16 = 1.9 \text{ kips/bolt}$$

$$f_{V_2} = \frac{M_e(2.75)}{J} = \frac{(31 \times 7)(2.75)}{2{,}120} = 0.3 \text{ kips/bolt}$$

$$f_{H_2} = \frac{M_e(15.0)}{J} = \frac{(31 \times 7)(15.0)}{2{,}120} = 1.5 \text{ kips/bolt}$$

$$f_R = \sqrt{(12.5 + 0.3)^2 + (1.9 + 1.5)^2} = 13.2 \text{ kips/bolt} < 14.9 \quad \textbf{o.k.}$$

Use: 16 1⅛-in. ϕ A325-F bolts

Gusset plate:

Provide ⁵⁄₁₆-in. oversized holes in gusset plate.

Try ½-in. plate thickness:

At Section 1-1 (Fig. 3-15):

$$\text{Effective width} = 2(14.0 \times \tan 30°) + 5.5 = 21.7 \text{ in.}$$

$$f_a = \frac{295}{2(21.7 \times 0.50)} = 13.6 \text{ ksi} < 22.0 \text{ ksi} \quad \textbf{o.k.}$$

At Section 2-2 (not shown):

Taken at end of girder. By comparison with Sect. 1-1 stress is low.

At Section 3-3 (Fig. 3-15):

See Fig. 3.16 for forces acting on section.

$$C = (12.5/17.0)(295) = 217 \text{ kips}$$

$$F = (11.5/17.0)(295) = 200 \text{ kips}$$

$$T = 248 \text{ kips}$$

$$f_v = \frac{1.5(200)}{2(38.0 \times 0.50)} = 7.9 \text{ ksi}$$

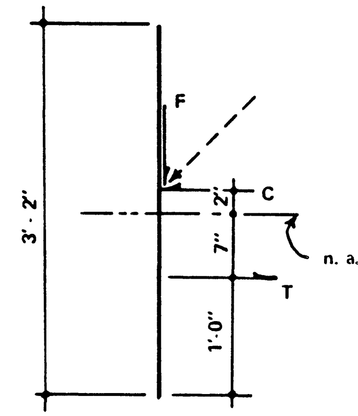


Figure 3.16

$$f_b = \pm\frac{(248 \times 7 + 217 \times 2)}{2(0.50)(38)^2/6} = \pm 9.0 \text{ ksi}$$

$$f_t = \frac{+248 - 217}{2(38 \times 0.50)} = +0.8 \text{ ksi}$$

$$f_t \pm f_b = +0.8 \pm 9.0 = -8.2 \text{ ksi and } +9.8 \text{ ksi}$$

$$F_v = 14.5 \text{ ksi} > 7.9 \text{ ksi} \quad \textbf{o.k.}$$

F_a depends on Kl/r of plate between lateral supports.

$$\frac{Kl}{r} \approx \frac{1.0 \times 18}{(0.29 \times 0.50)} = 124$$

$$F_a = 9.7 \text{ ksi} > 8.2 \text{ ksi} \quad \textbf{o.k.}$$

$$F_t = 22.0 \text{ ksi} > 9.8 \text{ ksi} \quad \textbf{o.k.}$$

Use: ½-in. plate thickness

SECT. 4: BUILDING CODE VARIATIONS FROM THE 1974 SEAOC SEISMIC CODE

All of the California state and local building codes have adopted the SEAOC seismic code with only minor changes, except in two design areas: the assignment of factors for critical facilities and the requirement for a dynamic design for certain structures. It is the requirement for a dynamic design which gives the design engineer the most trouble.

A dynamic seismic design has four phases:

1. The establishment of a design ground motion.
2. The determination of what response damping to assume and what stress and distortion performance is acceptable as a risk basis.
3. The determination of the dynamic response of the building to the design ground motion.
4. A stress and distortion analysis for the forces generated by the dynamic response.

Where the code allows the use of a code equivalent static force design, the code provides directly for Phases 1 and 3 through its static lateral force formulas. Phase 2 is generally covered by the code allowable stress criteria and the newly established seismic drift limitation. Phase 4 is generally left up to established engineering elastic analysis methods.

Though much attention is focused on Phase 3 of the dynamic design, it is really Phases 1 and 2 which are the most difficult for the designer at the present state of the codes. These phases are so difficult to codify at the present state of the art of seismic design that the codes have barely specified a general intent, let alone real definition.

Phase 1—For Phase 1, the building codes are requiring a seismology report which not only identifies any possible seismic site foundation hazards, such as the possibility of actual surface faulting through the site, soil liquefaction, etc., but also predicts the nature and intensity of possible ground shaking. A fault zone can be identified by strata discontinuities, and soil liquefaction potential can be detected from soil samples. These hazards can therefore be definitely covered by the geologist and soils engineer. However, the problem of predicting possible ground shaking at the site is another matter.

The prediction of ground shaking must be largely based on statistical records of past ground shaking, recorded in terms of Richter Magnitude and Mercalli type scales, and related to accelerograph records and the dynamic response spectra for these records. Predictions must also consider the performance history of structures subjected to earthquake ground shaking.

The identification of nearby active faults, predicting the future activity of the faults, and estimating site surface ground motions resulting from that fault activity provide a valuable refinement to site ground shaking predictions. There are, however, at least two schools of thought regarding the prediction of site ground surface shaking related to activity on a given fault. One approach is based on statistical records of attenuation of ground surface shaking with distance from the fault, the attenuation being related to the type and magnitude of the fault slip. The other approach analyzes only the vibration in the base rock as it travels horizontally through the base rock and then up through the vertical soil profile at the site. These two approaches can lead to widely different surface motion predictions.

A problem arises, of course, when a site seismology report predicts a surface ground motion whose characteristics and intensity do not fit into the statistical history of surface ground motions and their known effects. With such a report, a major conflict with established seismic design concepts must result, not between dynamic and static design methods, but solely as the result of seismicity estimates. If such a conflict does arise, the structural engineer, together with the building department and geotechnical consultant involved, should resolve the problem before design proceeds.

As an aid to evaluating and using the ground shaking predictions of a site seismology report, the structural engineer should require that it include at least the following data:

1. Response spectra which provide the response at several damping ratios (2%, 5%, 10%) to an ensemble of Maximum Credible site ground motions.
2. Response spectra which provide the response at several damping ratios (2%, 5%, 10%) to an ensemble of Maximum Probable site ground motions. The occurrence interval used in establishing the Maximum Probable earthquake spectra may be as established by the code or building department.
3. A statement in the report evaluating the seismicity of the site as compared to an average site in California (preferably by numerical factor). The client and the structural engineer should know whether the authors of the report think that this particular site is especially risky or safe, and where the authors fit into the entire field of seismicity prediction.
4. If the building department or the client requires a time-history analysis, an ensemble of predicted Probable and Credible ground motions is required. In order for the engineer to evaluate them, these ground motions must be presented as simulated records, not only of ground acceleration, but also of velocity and displacement, along with damped response spectra. All of these parameters are needed to test the credibility of the given ground motion.

It is obvious, of course, that a seismology report predicting unusually high ground motion intensities, or an unusual relation between peak response and building period, must be suspect. It is not tolerable to the structural engineer, the client, or the general public that buildings on similar adjacent sites be designed to widely different criteria based solely on seismological reports by different authors. Some degree of the uniformity of criteria imposed by codes on building design must be required of the site reports, if the end product building designs are to have any uniformity of risk provisions. This may suggest more than one site seismology report if the first does not seem to fit into the envelope of consensus opinion.

Phase 2—Since damping significantly affects the magnitude of both elastic and inelastic dynamic response, an estimated percentage of critical damping must be used with any dynamic analysis. It is not difficult to obtain experimental damping values from small ambient vibrations or from the similar small amplitude vibrations induced by shaking machines. However, damping is known to increase greatly with the amplitude of vibration, and it is the large amplitude response to strong ground motions which is critical. It is not feasible or desirable to produce these large amplitude response motions in actual buildings by the use of test machines. The data available on the damping associated with large amplitude vibration has been obtained from small test frames and from accelerograph records made in buildings subject to strong earthquake shaking. This data is sparse; however, records from a number of instrumented buildings subjected to strong shaking indicate that an analysis based on an assumed viscous damping of 5% for all modes gives good agreement with the recorded response of steel moment frame buildings. Interpolating and extrapolating available damping data generally indicates values from 2% to 10%, depending on how extensive the yielding is expected to be. Specifically, the damping for steel braced frames might be estimated at 2%, steel moment frames at 5%, and concrete shear walls at 2% to 5%. This seems to be the general order of assumed damping, though there is no close agreement on "best estimates."

The codes do not specify damping values because of the lack of available test data. The damping to be applied in establishing the design response is subject to the approval of the building official.

The codes do not specify the acceptable level of stress and distortion associated with a dynamic response to site ground motion predictions. This is, at present, left to the Building Department to be approved on an individual basis, and there are no standards to provide consistent decisions. It has always been recognized that the elastic force levels involved in the dynamic response to strong ground shaking are so great that it is generally impractical to design a building to remain entirely elastic. It has further been recognized that properly controlled inelastic response is not hazardous, and is acceptable from both safety and damage standpoints. The problem at the present state of material, element, frame, and building testing is in defining properly controlled inelastic response.

The most common approach is to factor down the elastic response spectra by an assumed ductility factor which is related by judgment to the type of frame involved. That type of frame is therefore assumed to be competent to properly control the degree of inelastic response represented by the factoring. The stresses and distortions determined from an analysis for the factored down elastic response are generally required to be checked against the same code allowable stresses and distortions as for buildings not requiring a dynamic design.

Since the margin between allowable stresses and specified yield capacities varies with the material, element, and type of stress, and varies even more with actual designed and furnished yield capacities, the actual degree and type of inelastic response may vary widely when controlled by code stresses. For this reason some authorities require a reduced response factoring, used with ultimate capacities rather than working stresses. Since much of the code is set up only for working stresses, this requires the establishment of acceptable ultimate capacities for many materials and elements.

Arbitrary factoring of elastic responses to represent inelastic response adequately predicts forces only as they are limited by yielding. Factoring does not apply at all to distortions, since inelastic response distortions are generally of the same order as elastic response distortions. For this reason some authorities require that stresses and distortions be calculated for the unfactored elastic response. The acceptable ratio between computed stresses and yield (or ultimate capacity) stresses is specified. Distortion (drift) limits are set on a judgment basis related to the reduced drift accepted for factored responses.

Phase 3—Phase 3 of dynamic seismic design is the computation of a building's MDF response to a given base motion. The essential options, generally recognized by codes and building departments, for determining the response are:

1. The use of SDF input spectra to determine the response for the principal modes of vibration of the structure, combining the SDF modal responses or a probability and participation basis, and reducing the elastic responses on an arbitrary numerical basis to allow for inelastic response capacity.
2. The use of a time-history computer dynamic analysis for given ground motions to determine directly the MDF response of the structure by combining modal responses at every level at every interval of time. If the program is for an elastic response (constant frame stiffness), estimates of inelastic response must be based on arbitrary factoring, just as for the spectrum approach. However, the factoring can be applied directly to element stresses computed for the unfactored response, allowing a better feel for design than with pre-factoring.
3. The use of a time-history computer dynamic analysis by direct integration (not modal response) to determine directly the elastic and inelastic response of the structure based on a changing stiffness and checked by an energy balance program.

Dynamic designs are generally required by codes for dynamically irregular buildings, buildings housing critical facilities, and tall buildings. Since the code static force distribution has been shown to essentially envelope the real dynamic force distribution for a dynamically regular building, it may be expected that, for a regular building, the response forces for a dynamic analysis will differ from those of a code static design only because of input motion differences. The intensity of ground motion input for a dynamic design will, of course, greatly exceed those inferred from the factored down code static force response. However, the character of the response cannot be greatly different from the code assumptions without being in conflict with the code static approach.

If it is assumed that the same ground motion should be applied to an irregular building as to a regular building on the same site, then the same design spectrum characteristics should be assumed. Essentially, these assumed critical characteristics are: (1) Each *critical* ground motion will have an acceleration response which may peak for any system period less than about 0.5 sec. Therefore, for any modal period falling in the 0 to about 0.5 sec. range, the same peak acceleration response is assumed for all responses to a given intensity level of ground motion; (2) each *critical* ground motion will have a velocity response which may peak for any system period greater than about 0.5 sec. Therefore, for any modal period greater than about 0.5 sec., the same peak velocity response is assumed for all responses to a given intensity level of ground motion.

The application of the modal spectrum approach to dynamic design has been discussed in considerable detail in Sect. 2 as a basis for understanding the code response formula used with the code static method of seismic design. It can be seen from this discussion that, for a modal response analysis based on the above response spectra characteristics, the effect of the modal period relationships associated with an irregular building will have little effect on the base shear. However, the distribution of shear up the building, for an irregular building, will be quite different from that of a regular building.

A modal spectrum analysis determines the distributed response accelerations at each level and adds the resultant shears algebraically on a participation and statistical probability basis at each level. The shears are added, from the roof down to the base, to obtain the base shear. This is the reverse of the code static approach of calculating a base shear, then distributing the shear up the building on an assumed shear envelope basis.

In order to obtain the modal accelerations, the mode shape (the building distortion curve) for each mode of vibration must be calculated. Since the building will always vibrate in that shape for that mode of vibration, and the period of vibration will remain the same for that mode of vibration, the accelerations will vary up through the building according to the mode shape displacement variation. The relative maximum amplitude of the displacement of the modes decreases rapidly with the increase in their order (first, second, third, etc.). Therefore, the participation of the modes decreases rapidly with the increase in the order of the modes.[1]

This may seem highly complex and may seem to indicate a response whose parameters cannot even be estimated without a detailed analysis based on an accurate mathematical model of the building. The complexities may be kept in perspective, however, by referring to the standard seismic shear envelope. The shear envelope for a force at the roof is a rectangle. The shear envelope for an assumed triangular loading (with the base of the triangle at the top) is a parabolic area equal to two-thirds of the area of a rectangle. The normal assumed shear distribution is a combination of the two. If all of the horizontal force were assumed at the top, the effect on the shear envelope would be to fill in the top of the shear envelope to a full rectangle. Even this assumption would not result in a radical increase in the cost of the steel frame. It is not possible to get a more severe load distribution than that represented by a rectangular shear diagram.

All of the preceeding discussion is for the analysis of a given building frame. But before any analysis can be made, there must be at least a preliminary frame design to work on. Therefore, from a design standpoint, Phases 3 and 4 must be worked together, since the dynamic response depends on the building period, which depends on a design for the response force level. For the first trial, the fundamental period can be estimated according to the formulas given in the code for static equivalent seismic design. For this fundamental period, the SDF response acceleration can be obtained from the unfactored or factored site spectra. Using the first trial response shear coefficient ($C_1 = a/g$) and an assumed drift coefficient, an improved estimate of the fundamental period can be obtained with the formula $T = 0.25 \sqrt{\Delta/C_1}$. The displacement Δ is the top level displacement, in inches, obtained by multiplying the assumed drift coefficient by the height from the base level to the top level.[2] The code limit drift of 0.5% can be assumed for a steel moment frame designed to a site response which has the usual factoring for ductility. For steel braced frames or shear wall systems, a drift of 0.2% to 0.5% may be estimated, depending on the braced frame or wall height-to-length ratio. When the fundamental period and SDF response are made to be reasonably compatible, a preliminary frame design can be computed for the SDF response. An MDF response can then be obtained for the trial frame. The rest of the design will be involved in modifying the frame by trial and error.

An analysis of the many factors affecting drift control for moment frames is given in Sect. 6. It is important for good design to know and consider all of these factors; however, for preliminary design, considerations of girder and column bending on a frame centerline basis will reasonably approximate the drift obtained using clear span bending plus panel zone contributions. For unusual member depth-to-length proportions, this should be checked. Building chord drift is not usually significant for frame aspect ratios less than 2. For preliminary design calculations and for simple building flexibility checks, the flexibility can often be determined with sufficient accuracy using portal shear distribution assumptions and checking a single typical bay at a few stories. Simplified chord drift calculations should be made if the frame aspect ratio is greater than 2.

For buildings with shear walls or braced frames, the elastic flexibility and the rocking moment resistance of the walls or braced frames should be determined. In addition, the flexibility of the frames that will be distorted if the wall or braced frame starts to rock should be determined. Shear transfer to the frame at the maximum credible ground motion intensity should then be estimated on a rational, simple, if not a detailed, basis. Finally, both the capacity of the frame to resist the shear transferred to it and the dynamic flexibility at this most critical stage should be computed.[2]

Phase 4—This phase needs comment only in regard to consideration of inelastic load sharing and the many approximations involved in the computation of the response forces. It should be remembered that stress levels based on elastic responses which have been factored down to account for inelastic response do not really represent the state of stress to be expected during a strong ground shaking. Involved "exact" analyses are only relevant to as many significant figures as is justified by the input. With inelastic yielding, the exact distribution of stress amongst elements sharing a common load is seldom significant.

SECT. 5: SEISMIC DESIGN TERMINOLOGY (PART 2)

Ductility Measurement—Present measures of ductility are confused. Energy capacity demands of earthquakes are measured in terms of force times horizontal displacement, leading to a ductility measure defined as the ratio of yield level horizontal displacement to the maximum horizontal displacement required to meet a given energy demand. A more significant measure, as far as member and joint performance, is the ratio between the yield point strain for each of those elements and the maximum demand strain. If the element (joint or member) which develops a plastic hinge contributes only 20% of the horizontal drift, then a ductility factor of 2 by the first definition requires a ductility factor of 5 by the second definition. For instance, if a total horizontal displacement equal to 200% of the total elastic horizontal displacement must be provided by an element which contributes only 20% of the total elastic drift, that element must have a ductility factor of 100% ÷ 20%, or 5.

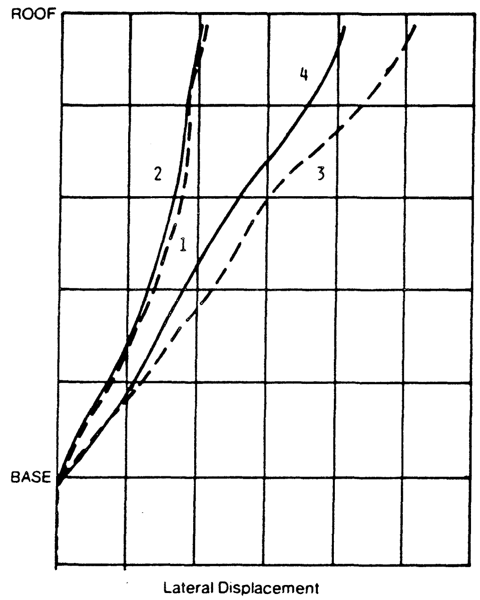
It seems that the significant factor in determining the inelastic capacity limit of a member or joint is the *actual rotation demand*, not the *ratio* between elastic and inelastic. From test frames, the potential for real failure can be evaluated if the hinge rotation demand is known. For instance, a 1% rotation will show little distress in a steel member or joint. If the drift to yield is deducted from the total drift demand, the yield rotation demand is obtained for whatever element develops a plastic hinge. This is defined by the demand drift coefficient less the yield point drift coefficient, since all of the yield drift results from joint rotation. The drift coefficient measures the joint rotation in radians, because it measures the tangent of a very small angle. Thus, a drift coefficient demand of 1% indicates a plastic hinge rotation demand of 0.57 degrees, if the elastic portion of the drift is neglected. If the elastic portion of the drift coefficient were 0.5%, the plastic rotation demand would be only 0.29 degrees.

Drift coefficients give the best measure of the actual ductility demands, since they measure strain demands which can then be related to test data.

Relative vs. Absolute Response Motions—Relative motions determine the distorted shape of, and the forces induced in, vertical frames. Absolute motions are the actual motions at any level, in reference to its original static position. Objects supported on that level will be subjected to a base motion which is the absolute motion of that level.

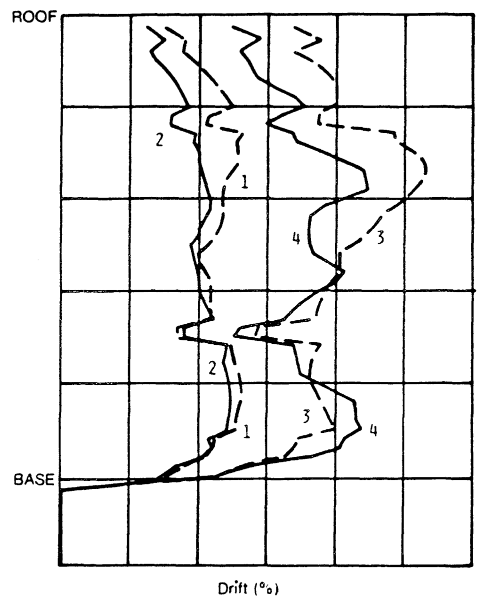
Design spectra all represent maximum *relative* motions and will not provide the absolute motions at any floor, or even their maximums. To obtain absolute motions, a time-history computer analysis for a given record must be run. Accelerographs record absolute motion. To obtain the motion of one level relative to another, the two recorded motions must be added algebraically at every instant in time.

Elastic Response vs. Inelastic Response—Most of the present dynamic computer building analyses are run on elastic response programs, that is, the frames are assumed to remain elastic, with stress and strain proportionately constant. These programs input a base motion and record the changing distortion of the building during the duration of the motion, and the associated building forces and motions involved. When the forces generated are greater than the elastic capacity of the structural frame, the results of an elastic analysis are not directly applicable. However, because of the complexities involved in an inelastic dynamic analysis, most computer programs so far developed for inelastic response are restricted to simple frames. Comparisons of inelastic response to elastic response for these simple frames and a few large frames indicate that the most important factor,



Computer dynamic analysis of a 50 story building.
Curves 1 and 3 elastic analysis
Curves 2 and 4 inelastic analysis

Figure 5.1



Computer dynamic analysis of a 50 story building.
Curves 1 and 3 elastic analysis
Curves 2 and 4 inelastic analysis

Figure 5.2

frame distortion, is generally predicted with reasonable accuracy by an elastic analysis, though the frame is forced past the elastic range.

See Figs. 5.1, 5.2, and 5.3 for results of the inelastic analysis of a 50-story building.

Inelastic Response—Elastic systems, and the elastic portion of inelastic systems, depend on storing the energy input to the building when the base is moved in one direction, and then releasing the energy during reverse motions. Inelastic systems absorb the energy in local yielding and plastic hinge action. The energy absorbed by plastic hinges in ductile frames can provide this required energy absorption.

Many non-structural elements will also absorb energy, at least until they break. This is cited not only as the reason why low capacity frames survive earthquakes, but also as the justification for under-designing structural frames. It does not seem either necessary or prudent to rely on this undependable type of resistance. It seems that all of the energy absorbing capacity needed to survive earthquakes can be provided in properly designed structural frames.

Energy Balance—When the ground moves, and thereby moves the base of a building, the integral, with respect to time, of the instantaneous inertia force of the building (its instantaneous base shear) times the instantaneous ground velocity represents energy input to the building. In terms of small increments of time, the energy input to the building during each time increment equals the base shear force at that time increment acting through a distance equal to the ground velocity at that time increment, times the small time increment. A computer can, therefore, be programmed to give a continuous record of energy input to a building throughout the time-history of ground shaking. The energy is added algebraically throughout the time steps. At any time when the base shear and ground velocity are acting in the same direction, the energy input will be negative. When they act in opposite directions, the energy input will be positive.

The building will store some of this input in the form of kinetic energy (due to the building's motion) and strain energy (due to the building's elastic distortion). The rest of the input energy will be dissipated in the form of damping and plastic yielding (large inelastic strains). The stored energy is added algebraically throughout the time steps, the energy stored in one direction being returned by motion in the opposite direction. The dissipated energy is not recoverable and is additive for yielding in either direction.

The energy involved in the movement of gravity loads through vertical distances is computed in a similar manner and added to the energy balance equation.

At any time during the ground shaking, the stored energy plus the dissipated energy must equal the input energy. For the exceedingly complex modeling and programming necessary for an inelastic analysis, this energy check is essential to test the accuracy and stability of the integration process used in the computer program.

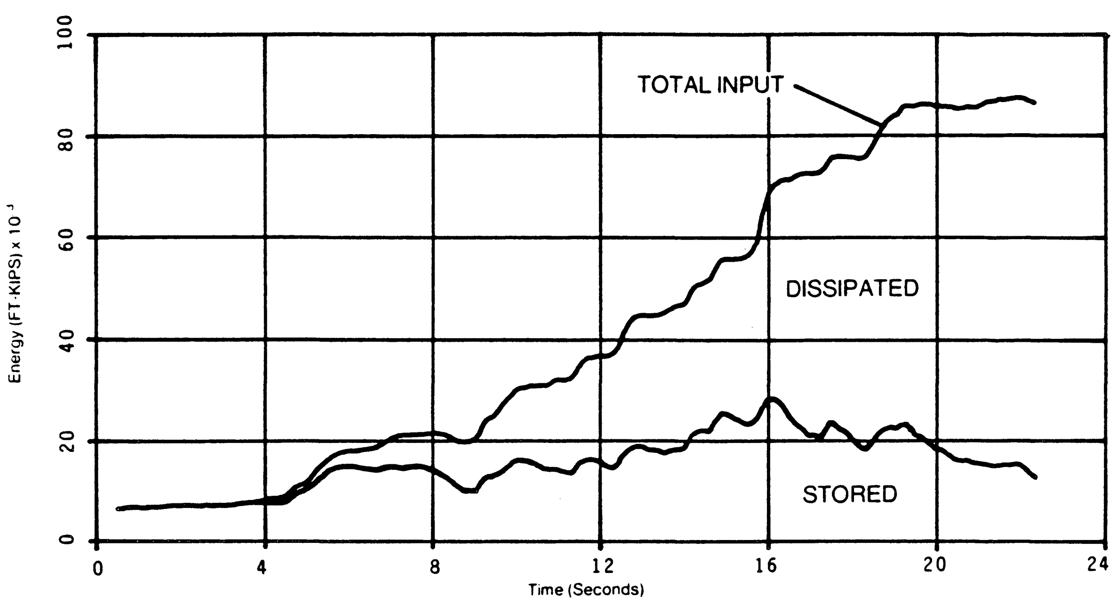


Fig. 5.3. Energy input to building during an earthquake

Inelastic Stiffness—Elastic response is derived by assuming a constant modulus of elasticity. While portions of a frame exceed the yield point (approximate proportional limit), the stiffness of the frame is changing and the problem of dynamic analysis becomes much more complex. To solve even small inelastic problems, the modulus of elasticity must be assumed to change according to some simple curve. The curve may be assumed (1) *elastoplastic*, i.e., the modulus suddenly becomes zero when the stress is above yield stress, (2) *bilinear*, i.e., the modulus suddenly reduces to a small quantity when the stress is above yield stress, or (3) a *Ramberg-Osgood function*, i.e., the modulus gradually falls off near and above the yield stress. See Fig. 5.4. The Ramberg-Osgood function closely represents the plots of load vs. strain for test members loaded gradually from zero up to yield point and into the inelastic range. The progressive yielding of members sharing a common lateral load (multi-column bent) is perhaps better represented by a bilinear stress strain relation.

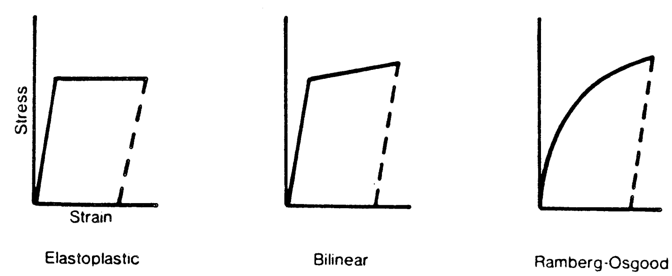


Fig. 5.4. Yield stress-strain assumptions

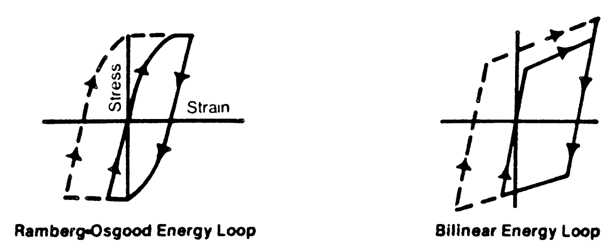


Figure 5.5

Hysteresis Energy Loops—For seismic testing, load is applied in one direction until a given strain is reached and then the load is reversed to a strain in the opposite direction. The loading is then cycled to determine an energy loop and to determine if the energy loop is stable or deteriorating. The area under the curve (within the loop) represents hysteresis energy absorption, because it represents force times distance. See Fig. 5.5.

If the plots for repeated loading cycles follow the first cycle plot closely, the stiffness and capacity of the test specimen is not deteriorating with load reversals. If the slope of the loop decreases, the stiffness is deteriorating. If the maximum ordinate decreases, the capacity is deteriorating. If the curve pinches in to include less area than a full wide loop, the energy absorbing capacity is reduced.

PΔ Effects and Instability—When a frame sways, a vertical load overturning moment develops which is given by the equation $M = P\Delta$. If this moment ever increased faster than the restoring force from the frame stiffness, instability would occur. For the vertical load problem, which is the concern of low earthquake risk areas, an instability threat is involved with extremely flexible frames. For these frames, if the frame is not stayed against sidesway and the vertical load is continuously increased, the frame will eventually buckle out from under the load. In other words, the P/A stresses plus the bending stress due to $P\Delta$ will reach yield and, since the load is constant, sidesway will continue until failure. The force necessary to stay the frame against sidesway is very small, the stiffness of the bracing generally being the critical factor. For vertical load stability, some nominal X-bracing or shear walls will provide the stiffness and strength needed for staying. If no X-bracing or shear walls are feasible, the design axial stress is reduced, the reduction being based on column and joint stiffness factors.

For frames designed to large lateral forces with controlled drift, the column strength reduction factors specified by codes for unbraced frames are not relevant, since the $P\Delta$ moments are effectively controlled.[3]

Real problems of instability due to $P\Delta$ effects in seismic test frames are few, and are connected only with large enough forces to cause extreme inelastic response. Definition of inelastic instability parameters for SDF systems have now been developed in terms of time and intensity of base motion, elastic strength, and stiffness.[4] For MDF systems, simple definition is probably not possible and only an inelastic dynamic analysis will identify problems. However, there is no indication that any problem will exist for properly designed structures with relatively uniformly distributed mass and stiffness.

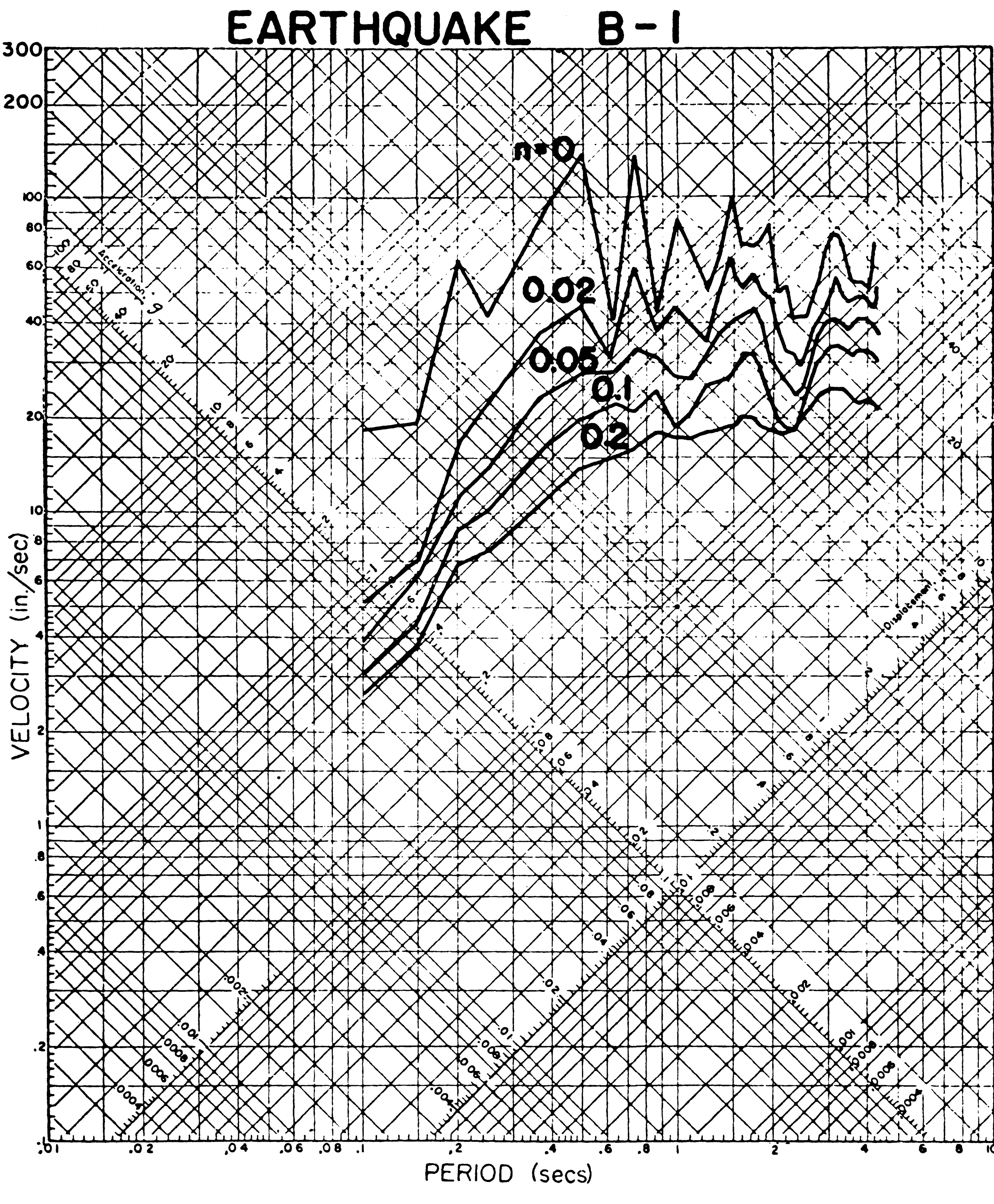
Earthquake Activity and Measurement—Earthquake shaking of the ground generally results from the passage of seismic ground stress waves that originate from the sudden release of stress by slip on a pre-existing geologic fault. Authorities see practically no probability of the creation of new faults, though, of course, many branch faults are still undiscovered. Very many old faults show no evidence of activity in geologically recent times.

Earthquake shaking is measured on three distinct types of scales: (1) Energy Release (Richter Magnitude), (2) Intensity of Effects (Mercalli type scale), and (3) Dynamic Spectral Intensity (dynamic response computed from recorded ground motion acceleration history).

The Richter Magnitude of an earthquake reflects the *total amount of energy released* by the fault slip. This scale is based on the logarithm of the maximum motion amplitude recorded on a standard seismograph, corrected to a distance of 100 kilometers from the epicenter. The scale is independent of the earthquake's effect on structures, though its possible effects may be estimated. Earthquakes of Magnitude 1 through 4 are small shocks which might result in little or no damage. Shocks of Magnitude 5 or greater could cause varying degrees of damage, depending on the character of the ground motion and the resistance of structures. Statistics indicate an upper bound of about Magnitude 8.5 for California and 9.0 for the world. Statistical studies of earthquakes show that it takes a long fault slip to generate a large Richter Magnitude earthquake (approximately 500 to 600 miles for an 8.5 rating). The ground shaking is most intense in the general vicinity of the fault slip and the intensity decreases generally with distance from the fault. Since this is an energy release scale, it is more a measure of the area shaken and the duration of strong ground motion than the intensity of shaking. The intensity of ground motion increases with the Richter Magnitude, but not on a logarithmic scale, and not necessarily directly. The ground shaking near the epicenter of a 7.0 thrust fault earthquake might be as great as that near the epicenter of an 8.0 strike-slip fault earthquake.

There are many "Intensity of Effects" scales, though the scale most widely used in the United States is the Modified Mercalli Scale. This type of scale measures the subjective observations of persons in the area shaken by the earthquake, as related to sensations of motion and effects on structures and other physical things. The scale is given in roman numerals from I to XII.

The Richter and Mercalli scales do not provide the detailed ground motion history needed to determine the actual intensity and character of ground motion. For measurement of Richter Magnitude, instruments (seismographs) which detect earthquakes at great distances are too sensitive to record strong motion close to the instrument. Strong motion instruments (accelerographs) are needed to record site ground accelerations as a function of time. The recorder for these instruments operates when triggered at a preset motion intensity, records while the strong motion lasts, and then stops until triggered again. This allows a detailed record to be made, with servicing only after a strong shake. This instrument gives a detailed record of ground shaking which can be used in the dynamic design of structures, and gives a detailed record of the building motion (when it is located in the upper floor of a building) to check actual dynamic response predictions. The checks made after the 1971 San Fernando earthquake provided good agreement between the recorded building motions and the motions predicted by dynamic analysis programs from the motions at the base of the building. Computed building periods and damping estimates related to strong motions were closely verified.[9,10]



LOG TRIPARTITE PLOT
Damping Values Are 0, 2, 5, 10 and 20 Percent of Critical

Figure 5.6

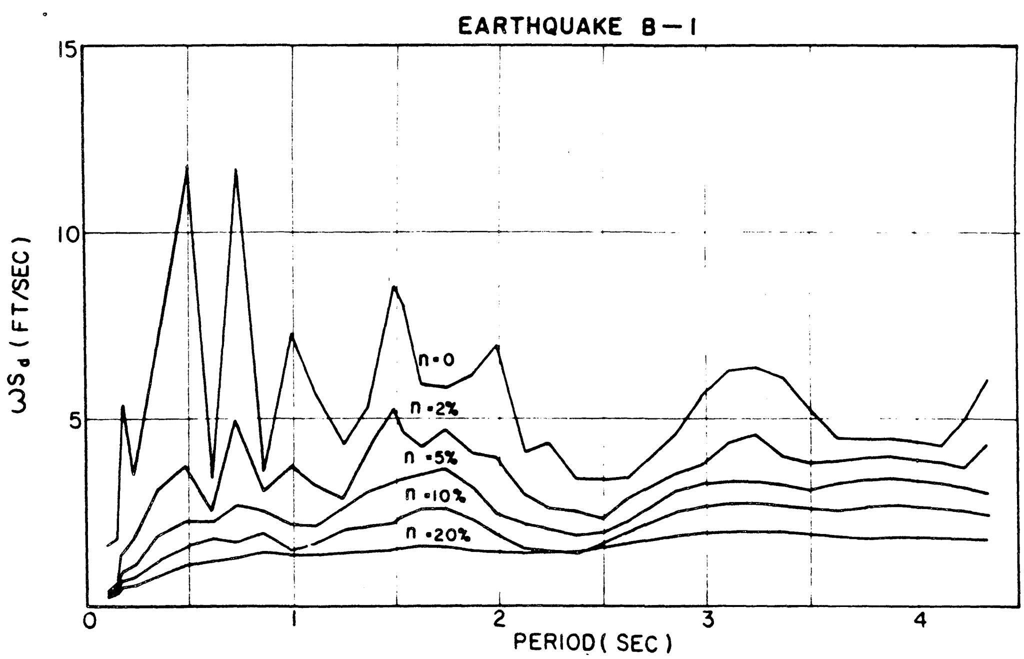
SDF response spectra at various percentages of critical damping, computed for the ground motions recorded by accelerographs, define the dynamic character and intensity of the motion. The spectral intensity is often measured by the average velocity response over a structural period range from about 1 sec. to about 4 sec. It may also be measured by a smoothed spectra covering all periods usually encountered in structures, though this does not provide a comparative index.

Log Tripartite Spectra Plots—Because of the SDF relationships between acceleration, velocity, and displacement, all three of these motion values can be shown by one curve on a log tripartite graph. This is an obvious mathematical graphing convenience, since three separate graphs are needed with normal rectangular coordinates. It should be noted, however, that the log scale radically condenses important areas of the period scale while radically expanding areas of little or no concern. The effect is to tend to conceal important information related to important periods and to overemphasize trends in areas which have little practical application. The engineer should also judge for himself whether he can better visualize the rate of response change when it is shown on a log or on a standard coordinate plot. He is at a disadvantage in judging the credibility and significance of any motion plot if he cannot picture clearly the way that the response is shown to vary with building period. See Figs. 5.6 and 5.7 for a comparison of plots.

Fourier Amplitude Spectra—An alternate method of analyzing the frequency content of a time-history record is afforded by the use of Fourier transforming to generate what are known as Fourier Amplitude Spectra relating the Fourier Amplitude in terms of any parameter, say velocity, to that of frequency. Modal frequencies contained in a time-history record appear as peaks in a Fourier Amplitude Spectra plot or a Power Spectra which is simply a Fourier Amplitude. The width of the peaks can be used to identify the amount of damping at the modal frequency associated with the peak. Mode shapes may also be determined for a structure and/or soil system using Fourier Spectra derived from a set of time signals located at stations in the structure. Generally speaking, Fourier Amplitude Spectra tend to give more precise indications of frequency content than do response spectra, especially in cases when the exciting force frequency spectrum is flat.

Earthquake Ground Motions—It is useful to define three levels of earthquake ground motion in regard to seismic design:

Probable Maximum Earthquake Motion—This represents the maximum general level of intensity which it is believed can be associated with *significant* probability of occurrence for a given time period. This means that there is enough probability of this intensity of motion to require the design of a structural frame whose response



RELATIVE VELOCITY RESPONSE SPECTRUM
Damping Values Are 0, 2, 5, 10 and 20 Percent of Critical

Figure 5.7

to that motion will be *very* predictably adequate. Considering the risks involved, this probability does not need to be very great.

Maximum Credible Earthquake Motion—This represents the maximum motion intensity which can be predicted as credible at a given site according to presently available data and theory. A structural frame should be able to survive this motion with *reasonable* predictability. Predictability is, of course, a matter of how far we extend our predictions based on presently available dynamic analytical data and physical test analogies.

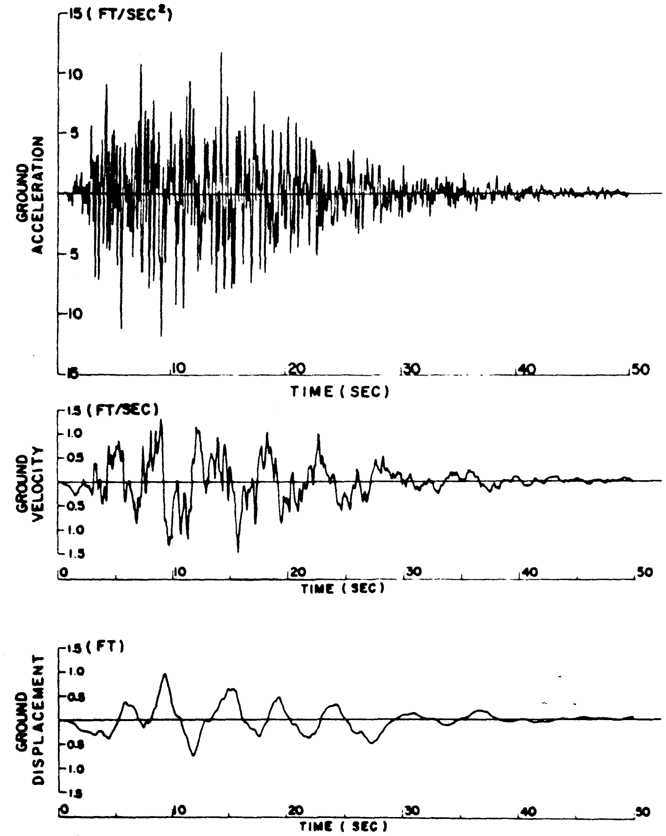
Maximim Possible Earthquake Motion—No one can say positively what maximum possibilities exist beyond our current theory and knowledge. It is assumed that frame capacity above that which is reasonably predictable will go a long way toward insuring a building's survival for motion intensities beyond those which are considered really credible.

Recorded and Simulated Earthquake Motions—The intensity of motion represented by all earthquake motion records obtained up to this time is, of course, less than the "maximum credible" value. California Institute of Technology and others have added to these records a series of simulated records based on theory, as an aid in more fully defining credible ground motions.[5]

Accelerograph Records—Accelerographs record acceleration, and the records must be integrated once for velocity, and twice to get displacement. To perform the integration on a digital computer (the only practical way), the records must be digitized by hand or by an electronic scanning device. The interval of digitizing is set at a practical level (for computer application costs) to obtain an essentially accurate reading of the scribed record. At any reasonable interval, some very short spikes of acceleration may be jumped, but if the spike is very short, so is its energy, and essential accuracy is maintained even though the accuracy is not absolute. There is no "base line" marking the exact change from positive to negative acceleration on the scribed record. The "base line" must be established statistically by one or another approximate method.[6] Several methods have been used, but one is standard since the 1971 San Fernando earthquake. It can be seen that the scribed record must be processed before it can be operated on by digital computer programs. The "Berg" and the "new" El Centro record represent two "base line" digitized versions of the same scribed record. See Fig. 5.8 for an example of ground motion characteristics.

Finally, it must be remembered that each recorded motion is a unique motion, never again to be exactly repeated. It is of little use spending a great amount of time fixing the exact record or dwelling on its exact spectra, except for the needs for analyzing past building performance. Recognition of its general characteristics and the position of the record in the envelope of growing statistical data are important. It is believed that this envelope of data, which includes motions such as those statistically generated by Cal Tech, is at this time the best probability estimate for future earthquakes. Any theoretically generated ground motion which does not generally fit into this envelope is highly suspect. Most engineering oriented authorities agree with this concept, while some geologically oriented authorities are supporting very theoretically generated ground motions, regardless of the relation of these motions to past records of ground motions and their effects on buildings.

Maximum Capacity—New definitions for maximum capacity, ultimate capacity, failure, etc., are needed. Common usage has led to thinking of these terms as a limiting condition in regard to collapse. In most cases these terms really mean a limit to some certain condition such as stress-strain linear proportionality. Safety factor working stress design is based on keeping the



EARTHQUAKE B-1

ACCELERATION, VELOCITY, AND DISPLACEMENT

Figure 5.8

stress at working loads below some given safe percentage of these arbitrary limiting conditions. Design in general is working away from this somewhat irrational approach, and seismic design in particular cannot afford it. New upper limits have to be established. Until this is done on a general basis, designers will have to do their best to convert working stress values to capacity limits.

This is particularly difficult for bolt values, weld values, some stresses such as shear, and combined stresses. Assumptions must be rational, if reasonable and practical seismic designs are to be achieved. Direct factoring to bring working stresses up to yield stresses may be too conservative for some values, if rational, feasible design for real seismic forces is to be consistent with the risk probabilities involved with these large forces.

There is also a need to consider the capacity of a frame in terms of the full capacity of all of the members which must reach full plastic yielding before the maximum lateral capacity of the frame is reached. For example, if the columns in a story are critical to story shear, the story shear capacity is not reached when one portion of the column cross section reaches yield stress. That is the point of incipient yielding, but the full frame yield capacity is not reached until a full plastic hinge forms at the top and bottom of every column in the story. This is a much greater yield level which will help control response, not only because of increased force capacity, but also because the transition stage includes the stability control of elastic members along with the energy absorbing capacity of members with inelastic strain. On the other hand, much greater axial forces can be developed due to the rocking moments associated with these larger horizontal forces, and they should be provided for.

SECT 6: DRIFT CONTROL ANALYSIS FOR STEEL MOMENT FRAMES

Need for Seismic Drift Control—The low frequency of occurrence for strong earthquakes, as well as practical considerations, indicate that we cannot be overly concerned with the psychological effects of drift induced by seismic forces. However, drift control is essential in order to insure structural integrity and to minimize non-structural damage. Any reasonable control of drift will insure structural stability, even in the inelastic range, but tighter drift control is needed to insure the structural integrity of joints and connections. Our still meager range of testing in the inelastic range all indicates that the integrity of joints made up even of such a ductile material as structural steel is only assured if the distortions are not too great. On the other hand, recent experience with structural steel indicates that a number of possible joint defects may be related to the thickness of the members and the weld sizes necessary to develop the members. Therefore, framing systems should be used which provide the needed drift control with the least possible member thicknesses.

Minimizing non-structural damage is not only an important economic consideration, but is in many cases essential to assuring life safety. One of the serious potential earthquake dangers is that of peeling off the architectural facade from the structural frame because of excessive frame distortion. Partitions and ceilings are very vulnerable to distortions and their collapse may injure people directly or may block the use of essential life safety services. Elevator shafts are also vulnerable to distortion damage and needed for life safety services.

It should be clear that adequate drift control is essential to the proper performance of all buildings. It should also be clear that the manner in which drift control is accomplished is exceedingly important, not only to the economy of the building frame, but also to the integrity of the joints which make up the frame.

Factors in Drift Control Design of Steel Moment Frames—The three basic components of drift are:

1. Column bending due to flexure and shear stresses.
2. Joint rotation due to girder and joint stresses.
3. Bending of the frame as a whole due to column axial deformation.

The first two components are commonly referred to as *shear drift.* This shear should not be confused with shear stress. It relates to shear type deflections considering the frame as a whole. The third component is referred to as *chord drift.*

The combined action of the first two components, or shear drift, is illustrated in Fig. 6.1. The total shear drift is the sum of Δ_c (due to column bending) and Δ_j (due to joint rotation). If the basic line of the girder were not horizontal, a third component of drift would be introduced. This would be the chord drift component.

The drift component due to column bending is clearly isolated in Fig. 6.2 by considering that the girders and joints are infinitely rigid. The formulas for column drift due to flexure and shear stresses are well known, although the second term, accounting for shear stresses, is generally neglected.

For emphasis, the effect of each of the factors included in the drift component formulas will be singled out for each of the drift components. For this purpose, it is assumed that all other factors remain constant, while only the factor being examined is varied. However, it is not intended to imply that these factors are independent from each other or from other building design influences.

Column Bending—The factors affecting drift due to column bending are:

1. Drift contribution is proportional to the cube of story height.

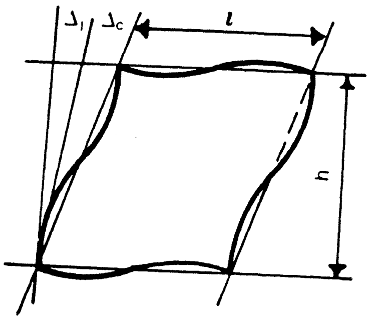
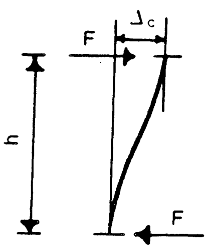
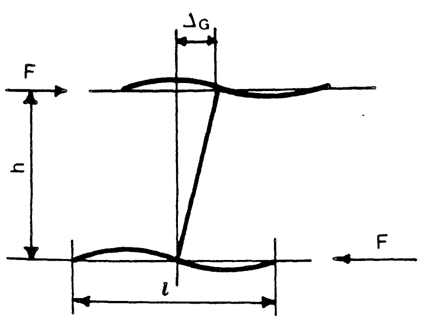


Figure 6.1



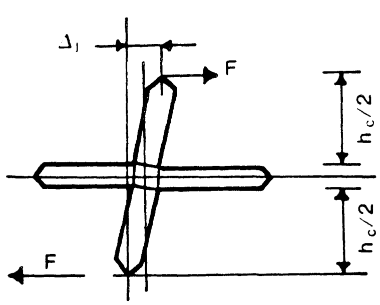
$$\Delta_c = \frac{Fh^3}{12EI_c} + \frac{2.5Fh}{A_wE}$$

Figure 6.2



$$\Delta_G = \frac{Flh^2}{12EI_G} + \frac{2.5Fh^2}{lA_wE}$$

Figure 6.3



$$\Delta_I = \frac{F}{0.4E} \times \frac{h_c^2}{Vol._w}$$

Where: $Vol._w$ = Volume of column web panel zone

Figure 6.4

2. Drift contribution is reduced by girder depth effect. The reduction factor is $(h_c/h)^2$.
3. Drift contribution is inversely proportional to the sum of the moments of inertia of columns sharing seismic shear.
   a. Column moment of inertia varies approximately as the square of the column depth.
   b. Column moment of inertia varies directly with weight of column per foot of length.

Joint Rotation—Drift due to joint rotation is clearly isolated by considering the columns to be infinitely rigid (see Fig. 6.3); only girder flexure-and-shear stresses will produce drift by allowing the joint to rotate according to the formula shown.

The factors affecting drift due to girder bending are:

1. Drift contribution is proportional to girder length.
2. Drift contribution is reduced by column depth effect. The reduction factor is $(l_c/l)^2$.
3. Drift contribution is proportional to the square of the story height.
4. Drift contribution is inversely proportional to the sum of the moments of inertia of girders sharing seismic shear.
   a. Girder moment of inertia varies approximately as the square of the girder depth.
   b. Girder moment of inertia varies directly with the weight of girder per foot of length.

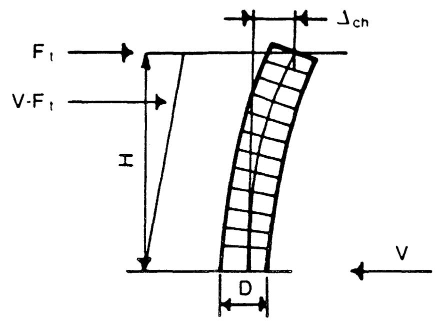
Joint rotation, however, results from several factors, not just girder bending. Joint rotation factors are:

1. Girder bending.
2. Joint web panel zone distortion.
3. Local column distortion at flange intersection.
4. Connection material slip or yielding.

The most important joint rotation factor, after girder bending, is joint panel zone distortion. This distortion is difficult to visualize, since the panel zone seems to extend past the actual joint area. A thorough analysis and considerable testing in recent years have shown that the panel zone does distort as an area isolated from the rest of the intersecting column and girder, as shown in Fig. 6.4.[7,8] There is no standard analysis or formula for the drift caused by this joint distortion, but the formula shown reasonably approximates this factor and agrees well with test results.

The factors affecting drift due to joint panel zone distortion are:

1. Drift contribution is inversely proportional to panel zone thickness, girder depth, and column depth.
2. Drift contribution is proportional to the square of the clear column height.



$$\Delta_{ch} = \frac{4F_tH^3}{3EAD^2} + \frac{4(V-F_t)H^3}{5.5EAD^2} \simeq \frac{VH^3}{EAD^2}$$

Figure 6.5

Miscellaneous "shear drift" factors are:

1. Drift is increased by unstiffened column flange.
2. Drift is increased by shear deflections in columns and girders.
3. Drift is increased by flexible connections.

Figure 6.5 illustrates the "chord drift" deflection caused by differential column axial deformations as a result of frame bending as a whole due to overturning (rocking) forces. It is sometimes thought that chord drift does not enter into problems of relative story displacement. However, in multi-bay frames, chord drift and shear drift are interdependent. Since floors must move as a whole, and center bays are involved in chord drift much less than end bays, they must be *more* involved in shear drift. The formula for chord drift can be approximated as VH^3/EAD^2, where H is the frame height, D is the frame length, and A is the effective area of columns resisting the chord drift.

Bending of Frame as a Whole—Factors affecting drift due to bending of frame as a whole (chord drift) are:

1. Drift contribution is proportional to the cube of the frame height.
2. Drift contribution is inversely proportional to the square of the frame length.
3. Drift contribution is inversely proportional to the weight of the column per foot of length.
4. Drift may be greatly reduced by flange or "tube" effect.
    a. "Tube" effect is inversely proportional (roughly) to the cube of the length of the flange wall spandrels.

It can be seen from these drift component percentages that all components can be important and design should consider them separately in relation to economical and effective drift control.

Summary of the Effects of Frame Dimensions on Drift—The effects of frame dimensions on drift can be summed up in terms of the various dimensions.

Story height affects all drift components:

1. Column drift component varies as the height cubed.
2. Girder drift component varies as the height squared.
3. Joint panel zone component varies as the height squared.
4. Chord drift component varies as the building height cubed.

Girder length affects girder and chord drift components:

1. Girder drift component varies directly as the length.
2. Chord drift component is reduced by the "tube" effect, which varies roughly as the girder length cubed.

Column and girder depths directly affect the column and girder drift components:

1. The components vary as the square of the depths.

Column and girder depths indirectly affect column and girder drift components:

1. Column drift component varies as the square of the ratio of clear story height to center line story height.
2. Girder drift component varies as the square of the ratio of clear girder length to center line length.

Frame length affects only chord drift:

1. This component varies as the square of the frame length.

Considerations for Other Frame Variables—The effects of other frame variables are also important:

1. All components of drift vary directly with frame member weight—except for joints and connections. The reduction of the drift component which is due to joint and connection distortion uses little material, though the connection costs must be carefully controlled.
2. Since a percentage increase in member unit weight will reduce the drift component involved by approximately the same percentage, the total weight is less if drift control concentrates on lighter members. Drift is often controlled with less total frame weight by reducing the girder component of drift, rather than by reducing the column component. This factor must be balanced, however, by inelastic dynamic response considerations, which generally favor keeping the columns elastic when frame yield strength is exceeded, and by joint welding considerations, which depend on girder flange thickness. Further, if panel zone doubler plates are

needed to control stress or distortion, it may be more economical to avoid doubler plates by using a heavier column section, since the column web thickness (basic joint panel zone thickness) for rolled sections varies with the section weight. Finally, the economy should be checked on the total weights of seismic frame girders vs. seismic frame columns, since the girders, though generally of lighter sections than the columns, generally have much greater total length. Drift control is cheaper near the top of the building where the frame is lighter, and this control also provides a favorable dynamic response by preventing whipping action.

3. In regard to total frame weight, the total weight of columns relates to story height, not only as the unit weight varies to control drift, but also in the total length of columns. Obviously, story height is much more important in the lower stories where the unit weight is greatest. High first stories and deep basement stories can increase total column weights enormously.
4. A final variable, and possibly the most important variable, is the number of columns sharing the total story shear. More columns mean more joints, but the joints involve less flange welding because the girder flanges are thinner. If there are more columns in a frame, the girders are shorter, thus reducing the girder drift component. "Tube" action, as has been noted, is entirely dependent on close column spacing in the flange frame. In terms of frame economy, the sum of these variables generally favors close column spacing. However, the most important factor favoring more columns to share the seismic load is the effect on the structural integrity of the frame. The most important protection against material and fabrication deficiencies is redundancy. Perfection can never be assured in an economical production material, in fabrication, or in inspection, but if there is enough redundancy, perfection is not needed. Any few deficiencies can be supported by the many competent joints of a lateral force resisting moment frame. And the chance of getting deficient joints is reduced if the member thicknesses involved in the joint are reduced because the load is shared by more members.

Fitting Frame Structural Design into the Whole Building Picture—When the essential facts of all of the preceding structural discussion have been inserted into the whole building design picture, there emerges an optimum framing system considering not only structural, but architectural and mechanical requirements as well. For buildings with large height-to-plan-area ratios, this often indicates a "tube" lateral framing system with all of the lateral force taken in the perimeter frames, unless the building height-width ratio in the transverse direction demands interior transverse frames to supplement the tube in the lower stories. Perimeter framing allows close column spacing, as well as deep girders and columns, without conflict with mechanical space requirements. It allows uniaxial moment joints, and column bending only on the strong stiff axis. It minimizes structural conflicts with mechanical requirements in the important distribution area around the building service core. And it allows maximum freedom in space planning of the most important building efficiency area, the service core. Maximum torsional resistance is also provided.

Effects of Frame Design on Joint Design and Performance Predictability—It is important to recognize that when the optimum framing system and framing dimensions have been selected, the die has been cast for the building frame. For any given lateral force and drift criteria, the weight of the frame is set, the member thicknesses have been determined, and the critical joint factors are established. If the resultant joint dimensions indicate possible problems in fabrication due to plate thicknesses or member depths, there are very limited trade-offs possible to ease the problem. In general, stiff buildings require many frames or a few very stiff frames, and very stiff frames demand very stiff joints.

APPENDIX A: 1974 SEAOC CODE*

"SECTION 1—GENERAL REQUISEMENTS FOR THE DESIGN AND CONSTRUCTION OF EARTHQUAKE RESISTIVE STRUTURES"

(A) General

The proper application of these lateral force requirements, both in design and construction, are intended to provide minimum standards toward making buildings and other structures earthquake resistive. The provisions of this Section apply to the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, and other elements.

Every structure shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the base. The force shall be assumed to come from any horizontal direction.

Where prescribed wind loads produce higher stresses, such loads shall be used in lieu of the loads resulting from earthquake forces.

*"*Recommended Lateral Force Requirements and Commentary*", *published by the Structural Engineers Association of California, San Francisco, Calif., 1974.*

(B) Definitions

BASE is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

BOX SYSTEM is a structural system without a complete vertical load carrying space frame. In this system, the required lateral forces are resisted by shear walls or braced frames as hereinafter defined.

BRACED FRAME is a truss system or its equivalent which is provided to resist lateral forces and in which the members are subjected primarily to axial stresses.

DUCTILE MOMENT RESISTING SPACE FRAME is a moment resisting space frame complying with the requirements given in Sections 2 and 4.

ESSENTIAL FACILITIES are those structures which must be functional for emergency post-earthquake operations.

LATERAL FORCE RESISTING SYSTEM is that part of the structural system assigned to resist the lateral forces prescribed in Section 1(D).

MOMENT RESISTING SPACE FRAME is a vertical load carrying space frame in which the members and joints are capable of resisting forces primarily by flexure.

SHEAR WALL is a wall designed to resist lateral forces parallel to the plane of the wall.

SPACE FRAME is a three-dimensional structural system, without bearing walls, composed of interconnected members laterally supported so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

VERTICAL LOAD CARRYING SPACE FRAME is a space frame designed to carry all vertical loads.

(C) Symbols and Notations

The following symbols and notations apply to the provisions of this Section:

C = Numerical coefficient as specified in Section 1(D).

C_p = Numerical coefficient as specified in Section 1(G) and as set forth in Table 1-B.

D = The dimension of the building in feet, in a direction parallel to the applied forces.

δ_i, δ_n = Deflections at levels i and n respectively, relative to the base, due to applied lateral forces.

F_i, F_n, F_x = Lateral force applied to level i, n, or x, respectively.

F_p = Lateral forces on a part of the structure and in the direction under consideration.

F_t = That portion of V considered concentrated at the top of the structure in addition to F_n.

g = Acceleration due to gravity.

h_i, h_n, h_x = Height in feet above the base to level i, n, or x, respectively.

I = Occupancy importance coefficient.

K = Numerical coefficient as set forth in Table 1-A.

Level i = Level of the structure referred to by the subscript i.
$i = 1$ designates the first level above the base.

Level n = That level which is uppermost in the main portion of the structure.

Level x = That level which is under design consideration.
$x = 1$ designates the first level above the base.

N = The total number of stories above the base to level n.

S = Numerical coefficient for site-structure resonance.

T = Fundamental elastic period of vibration of the structure in seconds in the direction under consideration.

T_s = Characteristic site period.

V = The total lateral force or shear at the base.

W = The total dead load and applicable portions of other loads.

w_i, w_x = That portion of W which is located at or is assigned to level i or x, respectively.

W_p = The weight of a portion of a structure.

Z = Numerical coefficient related to the seismicity of a region.

(D) Minimum Earthquake Forces for Structures

Except as provided in Section 1(G) and 1(I), every structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act non-concurrently in the direction of each of the main axes of the structure in accordance with the formula

$$V = ZIKCSW \qquad (1\text{-}1)$$

[UBC (14-1)]*

The value of Z equals 1.0 for areas of highest seismicity.

The value of I equals 1.5 for essential facilities. For all others, I shall not be less than 1.0.

The value of K shall be not less than that set forth in Table 1-A.

The values of C and S are as indicated hereafter except that the product of CS need not exceed 0.14.

**Bracketed formula numbers (preceded by the letters UBC) are the 1976 Uniform Building Code designations for the formulas they follow. Section 2314 of the 1976 Uniform Building Code is essentially the same as Section 1 of the 1974 SEAOC Code; formulas in those sections are identical.*

W is the total dead load and applicable portions of other loads such as: partitions, permanent equipment, snow, and, in storage and warehouse occupancies, a minimum of 25% of the floor live load.

The value of C shall be determined in accordance with the formula

$$C = \frac{1}{15\sqrt{T}} \qquad (1\text{-}2)$$

[UBC (14-2)]

The value of C need not exceed 0.12.

The period T shall be established using the structural properties and deformational characteristics of resisting elements in a properly substantiated analysis such as the formula

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\,\delta_i^2\right) \div g\left[\left(\sum_{i=1}^{n-1} F_i\delta_i\right) + (F_t + F_n)\delta_n\right]} \qquad (1\text{-}3)$$

[UBC (14-3)]

where the values of F_i, F_n, F_t, δ_i and δ_n shall be determined from the base shear V distributed approximately in accordance with the principles of Formulas (1-5), (1-6), and (1-7), or any other arbitrary base shear with a rational distribution.

In the absence of a period determination as indicated above, the value of T for buildings may be determined by the formula

$$T = \frac{0.05h_n}{\sqrt{D}} \qquad (1\text{-}3A)$$

[UBC (14-3A)]

or, for buildings in which the lateral force resisting system consists of moment resisting space frames capable of resisting 100% of the required lateral forces and such system is not enclosed by or adjoined by more rigid elements tending to prevent the frame from resisting lateral forces, T may be determined by the formula

$$T = 0.10N \qquad (1\text{-}3B)$$

[UBC (14-3B)]

The value of S shall be determined by the following formulas but shall not be less than 1.0:

For T/T_s = 1.0 or less,

$$S = 1.0 + \frac{T}{T_s} - 0.5\left[\frac{T}{T_s}\right]^2 \qquad (1\text{-}4)$$

[UBC (14-4)]

For T/T_s greater than 1.0,

$$S = 1.2 + 0.6\frac{T}{T_s} - 0.3\left[\frac{T}{T_s}\right]^2 \qquad (1\text{-}4A)$$

[UBC (14-4A)]

T in Formulas (1-4) and (1-4A) shall be established by a properly substantiated analysis but T shall not be taken as less than 0.3 sec.

The range of values of T_s may be established from properly substantiated geotechnical data, except that T_s shall not be taken as less than 0.5 sec. nor more than 2.5 sec. T_s shall be that value within the range of site periods, as determined above, that is nearest to T.

When T_s is not properly established, the value of S shall be 1.5.

> *EXCEPTION:* Where T has been established by a properly substantiated analysis and exceeds 2.5 sec., the value of S may be determined by assuming a value of 2.5 sec. for T_s.

(E) Distribution of Lateral Forces

1. Regular Structures or Framing Systems. The total lateral force V shall be distributed over the height of the structure in accordance with the following formulas:

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (1\text{-}5)$$

[UBC (14-5)]

The concentrated force at the top, F_t, shall be determined by the formula

$$F_t = 0.07\,TV \qquad (1\text{-}6)$$

[UBC (14-6)]

F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 sec. or less. The remaining portion of the total base shear V shall be distributed over the height of the structure including level n according to the formula

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (1\text{-}7)$$

[UBC (14-7)]

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution on that level.

2. Setbacks. Buildings having setbacks wherein the plan dimension of the tower in each direction is at least 75% of the corresponding plan dimension of the lower part may be considered as uniform buildings without setbacks, providing other irregularities as defined in this Section do not exist.

3. Irregular Structures or Framing Systems. The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.

4. Distribution of Horizontal Shear. Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral force-resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design.

5. Horizontal Torsional Moments. Provisions shall be made for the increase in shear resulting from the horizontal torsion due to an eccentricity between the center of mass and the center of rigidity. Negative torsional shears shall be neglected. Where the vertical resisting elements depend on diaphragm action for shear distribution at any level, the shear resisting elements shall be capable of resisting a torsional moment assumed to be equivalent to the story shear acting with an eccentricity of not less than 5% of the maximum building dimension at that level.

(F) Overturning

Every structure shall be designed to resist the overturning effects caused by the wind forces and related requirements, or the earthquake forces specified in this Section, whichever governs.

At any level, the incremental changes of the design overturning moment, in the story under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of the shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided.

Where a vertical resisting element is discontinuous, the overturning moment carried by the lowest story of that element shall be carried down as loads to the foundation.

(G) Lateral Force on Elements of Structures

Parts or portions of structures and their anchorage shall be designed for lateral forces in accordance with the formula

$$F_p = ZIC_pSW_p \qquad (1\text{-}8)$$

[UBC (14-8)]

The values of C_p are set forth in Table 1-B. Where C_p is 1.0 or more, the values of I and S need not exceed 1.0. The value of the product IC_pS shall be not less than 0.50 for equipment required to remain in place and be functional in essential facilities. The distribution of these forces shall be according to the gravity loads pertaining thereto.

(H) Drift Provisions

1. Drift. Lateral deflections or drift of a story relative to its adjacent stories shall not exceed 0.005 times the story height unless it can be demonstrated that greater drift can be tolerated. The displacement calculated from the application of the required lateral forces shall be multiplied by $(1.0/K)$ to obtain the drift. The ratio $(1.0/K)$ shall not be less than 1.0.

2. Building Separations. All portions of structures shall be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces.

(I) Alternate Determination and Distribution of Seismic Forces

Nothing in these Recommendations shall be deemed to prohibit the submission of properly substantiated technical data for establishing the lateral design forces and distribution by dynamic analyses. In such analyses the dynamic characteristics of the structure must be considered.

(J) Structural Systems

1. Ductility Requirements

a. Force Factor. All buildings designed with a horizontal force factor $K = 0.67$ or 0.80 shall have ductile moment resisting space frames.

b. Tall Buildings. In zones of high seismicity, buildings more than one hundred and sixty feet (160′) in height shall have ductile moment resisting space frames capable of resisting not less than 25% of the required seismic forces for the structure as a whole.

c. Concrete Frames. In zones of high seismicity, all concrete space frames required by design to be part of the lateral force resisting system and all concrete frames located in the perimeter line of vertical support shall be ductile moment resisting space frames.

EXCEPTION: Frames in the perimeter line of vertical support of buildings designed with shear walls taking 100% of the design lateral forces need only conform with Section 1(J)1 d.

d. Deformation Compatibility. All framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load carrying capacity and induced moments due to $(3.0/K)$ times the distortions resulting from the required lateral forces. The rigidity of other elements shall be considered in accordance with Section 1(E)4.

e. Adjoining Rigid Elements. Moment resisting space frames and ductile moment resisting space frames may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

f. Frame Ductility. The necessary ductility for a ductile moment resisting space frame shall be provided by a frame of structural steel conforming to Section 4, or by a reinforced concrete frame complying with Section 2 of these Recommendations.

g. Braced Frames. All members in braced frames shall be designed for 1.25 times the force determined in accordance with Section 1(D). Connections shall be designed to develop the full capacity of the members or shall be based on the above forces without the one-third increase usually permitted for stresses resulting from earthquake forces. Members of braced frames shall be composed of ASTM A36, A440, A441, A572 (except Grades 60 and 65) or A588 structural steel; or reinforced concrete bracing members conforming with the requirements of Section 3(B) of these Recommendations.

h. Shear Walls. Reinforced concrete shear walls for all structures shall conform to the requirements of Section 3 of these Recommendations. For the calculation of shear stress only, all masonry shear walls shall be designed to resist 1.5 times the force determined in accordance with Section 1(D).

i. Framing Below Base. In buildings where K = 0.67 or 0.80, the special ductility requirements of Sections 2, 3, and 4 of these Recommendations, as appropriate, shall apply to all structural elements below the base which are required to transmit to the foundation the forces resulting from lateral loads.

2. Design Requirements.

a. Minor Alterations. Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made unless the building as altered meets the requirements of these Recommendations.

b. Reinforced Masonry or Concrete. In zones of high seismicity, all elements within the structure which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete.

c. Combined Vertical and Horizontal Forces. In computing the effect of seismic forces in combination with vertical loads, gravity load stresses induced in members by dead load plus design live load, except roof live load, shall be considered. Consideration should also be given to minimum gravity loads acting in combination with lateral forces.

d. Diaphragms. Floor and roof diaphragms shall be designed to resist the forces specified in Table 1-B. Diaphragms supporting concrete or masonry walls shall have continuous ties between diaphragm chords to distribute the anchorage forces specified in Section 1(J)3a into the diaphragm. Added chords may be used to form sub-diaphragms to transmit the anchorage forces to the main cross ties. Diaphragm deformations shall be considered in the design of the supported walls. (See Section 1(J)3b for special anchorage requirements of wood diaphragms.)

3. Special Requirements.

a. Anchorage of Concrete or Masonry Walls. Concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. The anchorage shall provide a positive direct connection between the walls and floor or roof construction capable of resisting the horizontal forces specified in these Recommendations or a minimum force of 200 pounds per lineal foot of wall, whichever is greater. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds four feet. In masonry walls of hollow units or cavity walls, anchors shall be embedded in a reinforced grouted structural element of the wall. (See Section 1(J)2d for the requirements for developing anchorage forces in diaphragms. See Section 1(J)3b for special anchorage requirements for wood diaphragms.)

b. Wood Diaphragms Used to Support Concrete or Masonry Walls. Where wood diaphragms are used to laterally support concrete or masonry walls the anchorage shall conform to Section 1(J)3a. In zones of high seismicity, anchorage shall not be accomplished by use of toe nails, or nails subjected to withdrawal; nor shall wood ledgers be used in cross grain bending. The continuous ties required by Section 1(J)2d shall be in addition to the diaphragm sheathing; the diaphragm sheathing shall not be used to splice these ties.

c. Pile Caps and Caissons. Individual pile caps and caissons of every building or structure shall be interconnected by ties, each of which can carry by tension and compression a minimum horizontal force equal to 10 percent of the larger column loading, unless it can be demonstrated that equivalent restraint can be provided by other approved methods.

d. Exterior Elements. Precast, non-bearing, non-shear wall panels or other elements which are attached to, or enclose the exterior, shall accommodate movements of the structures resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of cast-in-place concrete or by mechanical fasteners in accordance with the following provisions:

Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or $(3.0/K)$ times story drift caused by the required seismic forces; or one-fourth inch (1/4″) whichever is greater.

Connections shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. Inserts in concrete shall be attached to, or hooked around reinforcing steel, or other wise terminated so as to effectively transfer forces to the reinforcing steel.

Table 1-A. Horizontal Force Factor K for Buildings or Other Structures

Type or Arrangement of Resisting Elements	Value of K
All building framing systems except as hereinafter classified.	1.00
Building with a box system as defined in Section 1(B).	1.33
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames designed in accordance with the following criteria: 1. The frame and shear walls or braced frames shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames. 2. The shear walls or braced frames acting independently of the ductile moment resisting space frame shall resist the total required lateral force. 3. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force.	0.80
Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force.	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building.[1,2]	2.5
Structures other than buildings and other than those set forth in Table 1-B.	2.0

[1] See Sect. 1(J)1g for additional detail requirements.
[2] The torsional requirements of Sect. 1(E)5 shall apply.

Connections to permit movement in the plane of the panel for story drift shall be properly designed sliding connections using slotted or oversize holes or may be connections which permit movement by bending of steel.

Table 1-B. Horizontal Force Factor C_p for Elements of Structures

Part or Portion of Buildings	Direction of Force	Value of C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions, masonry fences.	Normal to Flat Surface	0.20
Cantilever parapet walls.	Normal to Flat Surface	1.00[1]
Exterior and interior ornamentations and appendages.	Any Direction	1.00[1]
When connected to or a part of a building: towers, tanks, towers and tanks plus contents, storage racks plus contents, chimneys, smoke stacks, and penthouses.	Any Direction	0.20[2]
When connected to or a part of a building: rigid and rigidly mounted equipment and machinery not required for continued operation of essential occupancies.[4]	Any Horizontal Direction	0.20[3]
When resting on the ground: tank plus effective mass of its contents.	Any Direction	0.12
Floors and roofs acting as diaphragms.	In the Plane of the Diaphragm	0.12[5]
Connections for exterior panels or elements complying with Section 1(J)3d.	Any Direction	2.00[1]

[1] The product of IC_pS need not exceed the tabulated value of C_p.
[2] When h_n/D of any building is equal to or greater than 5 to 1 increase value by 50%.
[3] For flexible and flexibly mounted equipment and machinery, the appropriate values for C_p shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed.
[4] The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery. The structure to which the equipment or machinery is mounted shall be capable of resisting the anchorage forces. See Section 1(G).
[5] Floors and roofs acting as diaphragms shall be designed for a minimum force resulting from a C_p of 0.12 applied to w_x unless a greater force results from the distribution of lateral forces in accordance with Section 1(E).

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REFERENCES

1. *Jennings, Paul* Spectrum Techniques for Tall Buildings *California Institute of Technology, 1969.*
2. *Teal, E. J.* Seismic Drift Control Criteria *Engineering Journal of the American Institute of Steel Construction, Vol. 12, No. 2, 2nd Quarter 1975.*
3. *Teal, E. J.* Frame Stability Under Seismic Loading *Proceedings of Column Research Council, 1973.*
4. *Husid, Raul* Gravity Effects on Earthquake Response of Yielding Structures *California Institute of Technology, 1967.*
5. *Jennings, P. C., G. W. Housner, and N. C. Tsai* Simulated Earthquake Motions *California Institute of Technology, 1968.*
6. *Trifunac, M.D.* A New Method for Zero Baseline Correction of Strong-Motion Accelerograms *California Institute of Technology, 1970.*
7. *Becker, Roy* Panel Zone Effect on the Strength and Stiffness of Rigid Steel Frames *University of Southern California, 1971.*
8. *Krawinkler, H., V. V. Bertero, and E. P. Popov* Inelastic Behavior of Steel Beam-to-Column Subassemblages *University of California, Berkeley, 1971.*
9. San Fernando, California Earthquake of February 9, 1971 *U.S. Department of Commerce, NOAA Peport, 1973.*
10. Post Earthquake Analysis of DWP Building *Structural Engineers Association of California Seismology Committee Report, 1972.*
11. Manual of Steel Construction, Seventh Edition *American Institute of Steel Construction, New York, 1973.*
12. *Popov, E. P. and R. M. Stephen* Cyclic Loading of Full-Size Steel Connections *University of California, Berkeley, 1970.*
13. Steel Connections/Details and Relative Costs *Steel Committee, American Institute of Steel Construction, Los Angeles, Calif., 1973.*
14. *Fisher, John W. and John H. A. Struik* Guide to Design Criteria for Bolted and Riveted Joints *Chapter 15, John Wiley & Sons, New York, 1974.*