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TECHNICAL MANUAL

SEISMIC DESIGN FOR BUILDINGS

DEPARTMENTS OF THE ARMY, THE NAVY, AND THE AIR FORCE
FEBRUARY 1982

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FOREWORD

This updated manual with revised seismic design provisions governs the design and construction of Army, Navy, and Air Force facilities and supersedes the April 1973 issue. Basic criteria are stated herein with augmentations and clarifications of the criteria. Also, commentary and design examples are included to provide comprehensive applications and guidelines for the seismic-resistant design of facilities. The organization of the manual has been revised to present the topics in a more orderly manner. The dynamic analysis approach for seismic design is not covered but its use is not precluded in this manual.

The basic criteria cited are the "Recommended Lateral Force Requirements and Commentary" as published by the Structural Engineers Association of California (SEAOC). The design concepts and applications for the design of: (1) supports for electrical, mechanical and architectural elements and (2) structures other than buildings, have been revised. The applications of essential, high risk and other occupancy type structures are included with the use of the importance factors vice high-loss potential and low-loss potential facilities in the 1973 issue.

The general direction for the revision of the manual was by a Department of Defense Tri-Services Seismic Design Committee, i.e., representatives of the Office of the Chief of Engineers, Headquarters, US Army; Naval Facilities Engineering Command, Headquarters, US Navy; and Directorate of Engineering and Services, Headquarters, US Air Force. Detailed development of the manual was under the direction of the Office of the Chief of Engineers, Washington, DC and the US Army Division Engineer, South Pacific, San Francisco, California.

Coordination was maintained with the Naval Facilities Engineering Command at Headquarters, Washington, DC, and Western Division, San Bruno, California; and US Air Force Civil Engineering Offices at Headquarters, Washington, DC, and Western Regional Office, San Francisco, California.

SI CONVERSION UNITS

In view of the present accepted practice for building technology in this country, common U.S. units of measurements have been used throughout this publication. In recognition of the position of the United States as a signatory to the General Conference on Weights and Measures, which gave official status to the International System of Units (SI) in 1960, the table below is presented to facilitate conversion to SI Units. Readers interested in making further use of the coherent system of SI units are referred to: NBS SP 330, 1972 Edition, The International System of Units; and ASTM E380-76, Standard for Metric Practice. For conversion of formulas used in reinforced concrete design, the reader is referred to ACI 318-77, Appendix D.

Table of Conversion Factors to SI Units

To Convert From	*To*	*Multiply By*
inch (in)	meter (m)	$2.54^{*} \times 10^{-2}$
in^2	m^2	$6.4516^{*} \times 10^{-4}$
in^3	m^3	1.6387×10^{-5}
in^4	m^4	4.1623×10^{-7}
foot (ft)	meter	$3.048^{*} \times 10^{-1}$
pound-force (lbf)	newton (N)	4.4482
lbf · ft	N · m	1.3558
lbf/ft	N/m	1.4594×10
lbf · in	N · m	1.1298×10^{-1}
lbf/in	N/m	1.7513×10^{2}
lbf/in^2 (psi)	pascal (Pa)	6.8948×10^{3}

*Exact value; others are rounded to five digits.

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No. 5-809-10
NAVFAC P-355
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No. 88-3, CHAPTER 13

DEPARTMENTS OF THE ARMY, THE NAVY,
AND THE AIR FORCE
Washington, DC, *15 February 1982*

SEISMIC DESIGN FOR BUILDINGS

*This edition supersedes TM 5-809-10/NAVFAC P355/AFM 88-3, Chapter 13, dated 17 April 1973.

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CHAPTER 1
GENERAL

1-1. Purpose and scope. *a. Purpose.* This manual prescribes criteria and furnishes guidance for the design of buildings, some structures other than buildings, mechanical and electrical equipment supports, and utility systems in areas subject to damaging earthquakes. These criteria apply to all elements responsible for design of military construction located in seismic regions. In overseas construction, where local materials of grades other than those herein are used, the working stresses, grades, and other requirements of this manual will be modified as applicable.

b. Scope. This manual is for guidance in the design of buildings and other structures that are generally regular in shape, size, and concept. Buildings and other structures that are highly irregular will require analysis that rely on greater application of engineering judgment and experience in seismic design. Dynamic analysis requirements and alterations or evaluations of existing structures are not covered in this manual.

c. Design Criteria. Preparation of seismic design will be in accordance with the criteria and design standards herein. Criteria and design standards covered in the agency manuals for ordinary or nonseismic design are applicable to seismic design except where overriding criteria are contained herein. The seismic design and detail requirements herein are from the provisions of the "Recommended Lateral Force Requirements and Commentary," 1975 edition, of the Structural Engineers Association of California, 171 Second Street, San Francisco, CA 94105, except as modified herein.

1-2. Organization of manual. The general provisions for seismic design are covered by chapters 2, 3, and 4. Chapter 2 provides an introduction to the basic concepts of seismic design; chapter 3 contains the seismic design provisions; and chapter 4 provides a guide to the implementation of the seismic design provisions. Chapters 5 through 8 are concerned with seismic design in relation to structural materials, elements, and components. Chapters 9 and 10 cover seismic provisions for nonstructural components such as architectural, mechanical, and electrical elements. Chapter 11 covers structures other than buildings, and chapter 12 gives some guidelines for designing for the effects of earthquakes on utility systems. The appendices provide examples of design calculations.

1-3. Preparation of project documents. *a. Design Analysis.* A design analysis conforming to agency standards will be provided with final plans. This design analysis will include seismic design computations for the stresses in the lateral force resisting elements and their connections, and for the resulting lateral deflections and interstory drifts. (*Note:* In Zone 1, if wind loads control the design, a complete seismic analysis is not required; however, the seismic detailing requirements will be provided as specified.) The first portion of the Design Analysis, called the Basis of Design, will contain the following specific information:

(1) A statement of the seismic zone for which the structure will be designed.

(2) A description of the structural system selected for resisting lateral forces and discussion of the reasons for its selection. If setbacks are involved, the application of setback design provisions will be established.

(3) A statement regarding compliance with this manual and the selected values of "K", "C", "S", "I", and "Z".

(4) Any possible assumed future expansion for which provisions are made.

b. Drawings. Preparation of drawings will conform to agency standards for ordinary construction with the following additional specific requirements for seismic construction.

(1) Preliminary drawings will contain a statement that seismic design will be incorporated. The Basis of Design submitted with these drawings will give full information concerning the seismic loads that will be used, and the assumptions that will be made in carrying out the seismic design.

(2) Construction drawings for seismic areas will include the following additional special information:

(a) A statement of the Seismic Zone and the "K", "C", "S", "I", and "Z" values will be added to the tabulation of design loads.

(b) A list of the portions of the structure for which design was controlled by wind load will be placed immediately below the statements concerning seismic design.

(c) Details of construction will be similar or equal to the typical seismic details shown in the various sections of this manual.

(d) Assumptions made for future extensions or additions.

(3) Site adaptation of standard drawings will in-

clude design revisions for the seismic area as required.

c. Specifications. Project specifications will be prepared in accordance with agency standards and practices for ordinary construction except that applicable seismic guide specifications or supplements will be used as appropriate.

d. Cost Estimates. The special provisions required for seismic design generally result in an increase in construction costs of 1 percent to 5 percent. The amount of this increased cost depends on the overall concept and configuration of the building system and the geographical location of the building site. In some cases, a small amount of additional reinforcing bars, anchors, stiffener plates, or weld material may be all that is required to provide for the seismic design provisions. However, in other ca where the basic concept or configuration of building does not provide an efficient system of lateral force resistance, the additional costs to provide seismic force resistance can be appreciable. In geographical locations where the local construction industry is not experienced with the special details of seismic resistant construction, the differential costs for seismic design will generally be greater than for those areas, such as California, where seismic design construction is in general use. For example, the premium for seismic construction will be higher for *reinforced* masonry, *ductile* reinforced concrete frames, and *ductile* structural steel frames in areas where these types of construction are not common.

CHAPTER 2
INTRODUCTION TO SEISMIC DESIGN

2-1. Purpose and scope. This chapter provides an introduction to the basic concepts of designing buildings to resist inertia forces and related effects caused by earthquakes. General guidance is given for the selection and use of proper structural systems.

2-2. General. An earthquake causes vibratory ground motions at the base of a structure and the structure actively responds to these motions. Seismic design involves two distinct steps: determining (or estimating) the forces that will act on the structure and designing the structure to resist these forces and to keep deflections within prescribed limits. Other hazards, related to site location, are discussed in paragraph 2-7.

a. Determination of Forces. There are two general approaches to determining seismic forces: an equivalent static force procedure and a dynamic analysis procedure. This manual illustrates the equivalent static force procedure. Dynamic analysis procedures are not within the scope of this manual, but some discussion of structural dynamics is included in this chapter in order to explain the rationale of the equivalent static force procedure that is used in this manual.

b. Design of the Structure. The development of an adequate earthquake-resistant design for a structure includes the following: (1) selecting a workable overall structural concept, (2) establishing member sizes, (3) performing a structural analysis of the members to verify that stress and displacement requirements are satisfied, and (4) providing structural and nonstructural details so that the building can perform as intended. The structural designer must visualize the response of the structure to earthquake ground motions and provide a design that will accommodate the distortions and stresses which will occur in the building. In certain cases, some elements cannot accommodate these stresses and distortions. Elements such as rigid stairs, rigid partitions, and irregular wings can be isolated in order to reduce the detrimental effects to the lateral force-resisting system.

2-3. Ground motion. The response of a given building depends on the characteristics of the ground motion; therefore, it would be highly desirable to have a quantitative description of the ground motion that might occur at the site of the building during a major earthquake. Unfortunately, there is no one description that fits all the ground motions that might occur at any particular site. The characteristics of the ground motion are dependent on the magnitude of the earthquake (i.e., energy released), distance from the source of the earthquake (depth as well as horizontal distance), distance from the surface faulting (this may or may not be the same as the horizontal distance from the source), the nature of the geological formations between the source of the earthquake and the building, and the nature of the soil in the vicinity of the building site (e.g., hard rock or alluvium). Although the fully accurate prediction of ground motion is not possible, the art of ground motion prediction has progressed in recent years such that design criteria have been established in areas where historical earthquake records and geological information are available.

2-4. Structural response. If the base of a structure is suddenly moved, as in the case of seismic ground motion, the upper part of the structure will not respond instantaneously but will lag because of inertial resistance and the flexibility of the structure. This concept is illustrated in figures 2-1, 2-2, and 2-3 by showing the motion in one plane. The stresses and distortions in the building are the same as if the base of the structure were to remain stationary while time-varying horizontal forces are applied to the upper part of the building. These forces, called inertia forces, are equal to the product of the mass of the structure times acceleration, or $F = ma$ (mass is equal to weight divided by the acceleration of gravity). Because the ground motion at a point on the earth's surface is three dimensional (one vertical and two horizontal components), the structures affected will deform in a three-dimensional manner. Generally, however, the inertia forces generated by the horizontal components of ground motion required the greater consideration for seismic design since adequate resistance to vertical seismic loads is usually provided by the member capacities required for gravity load design. For ordinary structures within the scope of this manual, the inertia forces are represented by equivalent static forces. However, buildings can be idealized by the use of simplified models that represent the dynamic characteristics of the structure. For special structures the idealized models are subjected to time-history, response-spectrum, or other dynamic analyses, and the re-

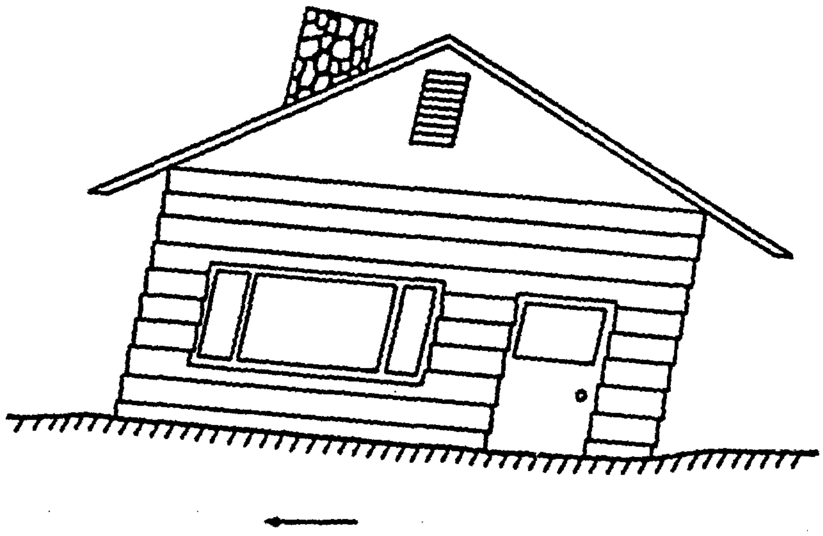

Figure 2-1. Schematic of Low-Rise Building Instantaneous Distortion During Ground Motion

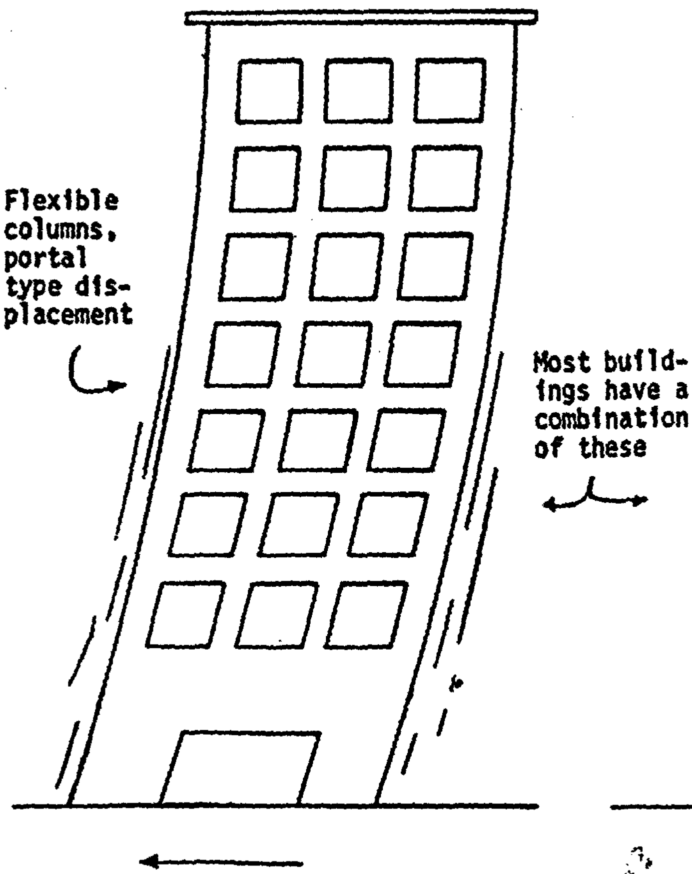

Figure 2-2. Schematic Showing Shear-Type Distortion

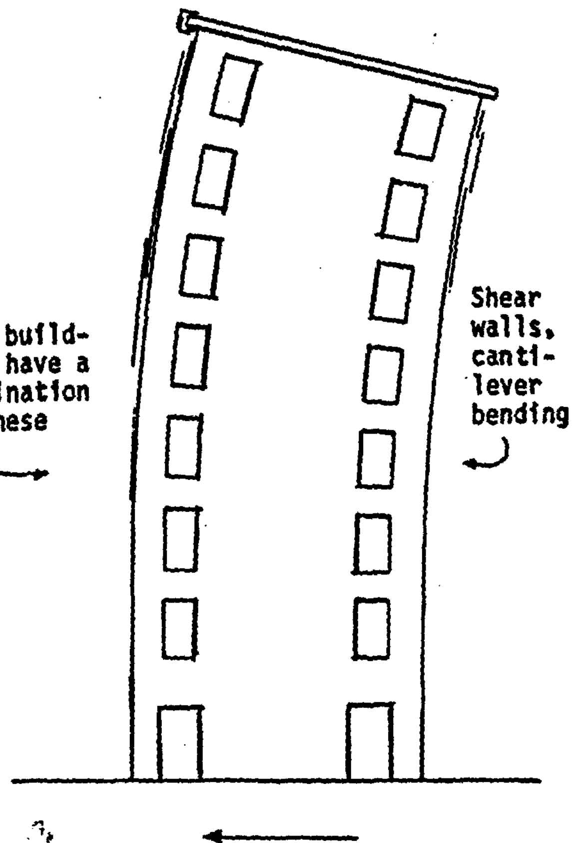

Figure 2-3. Schematic Showing Bending-Type Distortion

sults are used to determine the forces in the building.

2-5. Behavior of buildings. Buildings are composed of vertical and horizontal structural elements which resist lateral forces. The vertical elements that are used to transfer lateral forces to the ground are: (1) shear walls, (2) braced frames, and (3) moment-resisting frames. Horizontal elements that are used to distribute lateral forces to vertical elements are: (1) diaphragms and (2) horizontal bracing. Horizontal forces produced by seismic motion are directly proportional to the masses of building elements and are considered to act at the centroid of the mass of these elements. All of the inertia forces originating from the masses on and of the structure must be transmitted to the lateral force-resisting elements, to the base of the structure and into the ground. The path of these forces is discussed in chapter 4, paragraph 4-4*d*.

a. Demands of Earthquake Motion. The loads or forces which a structure sustains during an earthquake result directly from the distortions induced in the structure by the motion of the ground on which it rests. Base motion is characterized by displacements, velocities, and accelerations which are erratic in direction, magnitude, duration, and sequence. Earthquake loads are inertia forces related to the mass, stiffness, and energy absorbing (e.g., damping and ductility) characteristics of the structure. During the life of a structure located in a seismically active zone, it is generally expected that the structure will be subjected to many small earthquakes, some moderate earthquakes, one or more large earthquakes, and possibly a very severe earthquake. In general, it is uneconomical or impractical to design buildings to resist the forces resulting from the maximum credible earthquake within the elastic range of stress. If the earthquake motion is severe, most structures will experience yielding in some of their elements. The energy-absorption capacity of the yielding structure will limit the damage so that buildings that are properly designed and detailed can survive earthquake forces which are substantially greater than the design forces that are associated with allowable stresses in the elastic range. Seismic design concepts must consider building proportions and details for their ductility (capacity to yield) and reserve energy-absorption capacity for surviving the inelastic deformations that would result from a maximum expected earthquake. Special attention must be given to connections that hold the lateral force-resisting elements together.

b. Response of Buildings. A building is analyzed for its response to ground motion by representing the structural properties in an idealized mathematical model as an assembly of masses interconnected by springs and dampers. The tributary weight to each floor level is lumped into a single mass, and the force-deformation characteristics of the lateral force-resisting walls or frames between floor levels are transformed into equivalent story stiffnesses. Because of the complexity of the calculations for methods of dynamic analysis, the use of a computer program is generally necessary; these complex methods of analysis are generally used for critical structures. However, most buildings are designed by the equivalent static force procedure prescribed in this manual.

c. Response of Elements Attached to the Building. Elements attached to the floors of the building (e.g., mechanical equipment, ornamentation, piping, nonstructural partitions) respond to floor motion in much the same manner that the building responds to ground motion. However, the floor motion may vary substantially from the ground motion. The high frequency components of the ground motion tend to be filtered out at the higher levels in the building while the components of ground motion that correspond to the natural periods of vibrations of the building tend to be magnified. If the elements are rigid and are rigidly attached to the structure, the forces on the elements will be in the same proportion to the mass as the forces on the structure. But elements that are flexible and have periods of vibration close to any of the predominant modes of the building vibration will experience forces in a proportion substantially greater than the forces on the structure. For further discussion, refer to chapter 10.

2-6. Nature of seismic codes. Codes and criteria are established from the performance of buildings in past earthquakes. A code represents the consensus of a committee. Consensus means elements of compromise and generalized statements to cover uncertainties and limitations. Codes must of necessity be short and relatively simple; therefore, they do not account for all aspects of the complex phenomena of the response of actual structures to actual earthquakes. Seismic design codes provide a set of design static forces to represent the dynamic response of a structure subject to a complex earthquake ground motion.

a. Purpose. The basic purpose of a building code is to provide for public safety. The seismic provisions of this manual (chap 3) are based on the fourth edition of "Recommended Lateral Force and Commentary" of the Seismology Committee of the Structural Engineers Association of California (SEAOC). The introduction to the Commentary that publication is reprinted below:*

> "The SEAOC Recommendations are intended to provide criteria to fulfill life safety concepts. It is emphasized that the recommended design levels are not directly comparable to recorded or estimated peak ground accelerations from earthquakes. They are however, related to the effective peak accelerations to be expected in seismic events. More specifically with regard to earthquakes, structures designed in conformance with the provisions and principles set forth therein should, in general, be able to:
>
> 1. Resist minor earthquakes without damage;
> 2. Resist moderate earthquakes without structural damage, but with some nonstructural damage;
> 3. Resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage.
>
> In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. This, however, depends upon a number of factors, including the type of construction selected for the structure.
>
> "Conformance to the Recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum intensity earthquake. While damage in the basic materials now qualified may be negligible or significant, repairable or virtually irrepairable, it is reasonable to expect that a well-planned structure will not collapse in a major earthquake. The protection of life is reasonably provided, but not with complete assurance.
>
> "It is to be understood that damage due to earth slides such as those that occurred in Anchorage, Alaska, or due to earth consolidation such as occurred in Niigata, Japan, would not be prevented by conformance with these Recommendations. The SEAOC Recommendations have been prepared to provide minimum required resistance to typical earthquake ground shaking, without settlement, slides, subsidence, or faulting in the immediate vicinity of the structure.
>
> "Where prescribed wind loading governs the stress or drift design, the resisting system must still conform to the ductility, design and special requirements for seismic systems. This is required in order to resist in a ductile manner potential seismic loadings in excess of the prescribed loads."

b. Equivalent Static Force. The assumed equivalent total lateral force, equal to the base shear, is determined by the formula V = ZIKCSW (see chap 3 and 4 for seismic provisions). This approach attempts to recognize the available recorded experience and to some degree the qualitative dynamic analysis of simplified structures.

(1) The seismicity factor Z relates to severity of the ground motion at the site of the structure.

(2) The factor I represents the importance of the structure and is used to categorize the risk of damage to types of facilities.

(3) The factor K relates to the ductility and energy absorption qualities of certain types of structural framing systems, which historically have shown characteristic degrees of earthquake resistance.

(4) The product CS may be considered to be proportional to a response spectrum; however, it is applicable to base shear coefficients rather than spectral accelerations. The factor C accounts for the structural response as a function of the natural period and stiffness of structures. The coefficient S accounts for the variability of site conditions. Although CS is a function of the fundamental period of vibration, it is intended to represent the combined effects of all vibrational modes of the building.

(5) W is the weight of the structure.

(6) The total force V is distributed vertically along the height of a structure according to formulas that approximate the fundamental mode of vibration, with adjustments to approximate the effects of other participating modes of vibration.

c. Design Provisions. The seismic design provisions furnish a method for establishing the forces, describe acceptable basic systems, set limits on deformation, and specify the allowable stresses and/or strengths of the materials. The seismic design provisions are minimum requirements, and emphasis must be placed on structural concepts and detailing techniques as well as on stress calculations. The provisions are not all-inclusive ones: they work best for regular, symmetrical buildings. For unusual or large buildings, alternatives to the static provision•

that rely on dynamic analyses and/or greater application of engineering judgment and experience in seismic design are required.

2-7. Location of site. Site planning must consider geological, foundation, and tsunami (seawave) hazards as well as seismicity. Structures shall not be sited over active geologic faults, in areas of instability subject to landslides, where soil liquefaction is likely to occur, or in areas subject to tsunami damage.

a. Seismic Zones. The probability of the severity, frequency, and potential damage from ground shaking varies in different geographic regions. Regions with similar hazard factors are identified as seismic zones. The seismic zones prescribed by this manual are given in chapter 3, Design Criteria.

b. Fault Zones. Damage which is directly or indirectly caused by ground distortions or ruptures along a fault cannot be eliminated by design and construction practices; therefore, site planning must avoid these particularly hazardous locations.

c. Other Hazards. There are other hazards associated with earthquakes that should be considered. These include subsidence and settlement due to consolidation or compaction, landslides, and liquefaction. Liquefaction is a common occurrence in relatively loose cohesionless sands and silts with a high water table. The earthquake motions can transform the soil into a liquefied state as a consequence of the increase in pore pressure. This can result in a loss of strength in bearing capacity of the soil supporting a building, causing considerable settling and tilting. Also, this loss of strength can occur in the subsurface layer, causing lateral movement of surficial soil masses of several feet, accompanied by ground cracks and differential vertical displacements. These movements have severed pipelines and damaged bridges and buildings. There are several ways to stabilize the ground such as providing drainage wells, pressure grouting, or removing the liquefiable zone, but often the susceptible area is too extensive for an economical solution. The exposure to these hazards varies with the geography, geology, and soil conditions of the site, and the type of structure to be constructed. The professional judgment of geologists, soils engineers, and structural engineers shall be used to establish reasonable standards of safety.

d. Tsunami Protection. Each region along the Pacific Coast must be separately and carefully investigated for its tsunami-generation characteristics. Particular coastlines, inlets, and bays of the Pacific Ocean boundary are resonators of tsunami waves and may amplify the effects to large proportions. Assuming that tsunami warning services can ensure the safety of human life, there is as yet no hard-and-fast rule for establishing safety and economic standards. Where feasible, power plants, oil storage tanks, and other strategic facilities should be located on high ground, out of reach of high water. The methodology for predicting wave run-up is published in U.S. Army Engineers Waterway Experimental Station Technical Reports H-74-3, H-75-17, and H-77-16.

2-8. Selection of the structural system. It is of the utmost importance to make sure that the design efforts get off to a good start. Thus, it is essential that careful professional scrutiny be given to the design at its inception as well as at all significant stages of design development. The proper approach to be applied in the selection of a structural system that will achieve a reliable earthquake-resistant building must be based on performance criteria, alternative solutions, and corresponding costs.

a. Objective. The objective is to produce the structural system that is the most economical without compromising function, quality, or reliability. Final selection of materials and systems will be made with due consideration given to the cost of construction, architectural requirements, fire and other safety hazards, and maintenance and operating costs over the life of the facility. It is essential that the most efficient systems, methods, and materials be employed.

b. Economic Aspects. Usually, the major structural-architectural components of a building that have the greatest effect on the cost of construction are exterior walls, partitions, floor and roof decks, and the structural framing system. In some instances, the type of foundation may be a major factor in a cost study. Skillful planning, simple detailing, and arrangement of spaces to be compatible with repetitive modular construction all contribute greatly to reducing total building costs. On the other hand, the use of exotic or unconventional methods of construction may increase the costs and reduce the reliability of earthquake-resistance performance.

c. Planning Concepts. Participation of all disciplines of the design team in the conceptual planning and selection of basic construction materials will ensure the optimal design at lowest construction cost and minimize the total design effort. Procedures in the approach to develop a concept will vary depending upon the type of facility and the individuals on the design team.

2-9. Techniques of seismic design. For gravity loads, it has been a long-standing practice to design for strength and deflections within the elastic limits of the members. However, to control design within elastic behavior for the maximum expected horizontal seismic forces is impractical in high-seismicity areas (refer to para 2-5a). Hence, designers must resort to other techniques to achieve acceptable building performance (refer to chap 4, Design Procedures). A number of features contributing to seismic resistance are discussed below.

a. Layout. A great deal of a building's resistance to lateral forces is determined by its plan layout. The objective in this regard is symmetry about both axes, not only of the building itself but of the arrangement of wall openings, columns, shear walls, etc. It is most desirable to consider the effect of lateral forces on the structural system from the start of the layout since this may save considerable time and money without detracting significantly from the usefulness or appearance of the building.

b. Structural Symmetry. Experience has shown that buildings which are unsymmetrical in plan have greater susceptibility to earthquake damage than symmetrical structures. The effect of asymmetry will induce torsional oscillations of the structure and stress concentrations at re-entrant corners. Asymmetry in plan can be eliminated or improved by separating L-, T-, and U-shaped buildings into distinct units by use of seismic joints at junctions of the individual wings. Asymmetry caused by the eccentric location of lateral force-resisting structural elements, e.g., a building that has a flexible front because of large openings and an essentially stiff (solid) rear wall, can usually be avoided by better conceptual planning, e.g., by modifying the stiffness of the rear wall, or adding rigid structural partitions to make the center of rigidity of the lateral force-resisting elements close to the center of mass.

c. Irregular Buildings. Geometric configuration, type of structural members, details of connections, and materials of construction all have a profound effect on the structural-dynamic response of a building. When a building has irregular features, such as asymmetry in plan or vertical discontinuity, the assumptions used in developing seismic criteria for buildings with regular features may not apply. Therefore, it is best to avoid creating buildings with irregular features. For example, planners often omit partitions and exterior walls in the first story of a building to permit an open ground floor. This leaves the columns at the ground level as the only elements available to resist lateral forces, thus causing abrupt change in rigidities at that level. This condition is undesirable. It is advisable to carry all shear walls down to the foundation. When irregular features are unavoidable, special design considerations are required to account for the unusual dynamic characteristics (chap 4, para 4-4a(4)) and the load transfer and stress concentrations that occur at abrupt changes in structural resistance.

d. Lateral Force-Resisting Systems. There are several approved systems for the resistance of lateral forces (chap 3, table 3-3 and para 3-6; and chap 4, para 4-3c). All of the systems rely basically on a complete, three-dimensional space frame; a coordinated system of shear walls or braced frames with horizontal diaphragms; or a combination of the two systems.

(1) In buildings where a space frame resists the earthquake forces, the columns and beams act in bending (fig 2-4*a*). During a large earthquake, story-to-story deflection (story drift) may be a measure of inches without causing failure of columns or beams. However, the drift may be sufficient to damage elements that are rigidly tied to the structural system such as brittle partitions, stairways, plumbing, exterior walls, and other elements that extend between floors (para 2-9*i*). Therefore, buildings ' have substantial interior and exterior nonstructu damage, possibly approaching 50 percent of the total building value, and still be considered as structurally safe. While there are excellent theoretical and economic reasons for resisting seismic forces by frame action, for particular buildings this system may be a poor economic risk unless special damage control measures are taken (para 2-9*k*).

(2) A shear wall (or braced frame) building is normally rigid compared with a framed structure. With low design stress limits in shear walls, deflection due to shear forces (for low buildings) is negligible. Shear wall construction is an excellent method of bracing buildings to limit damage, and this type of construction is normally economically feasible up to about eight stories. Shear walls are usually of reinforced unit masonry, reinforced concrete (fig 2-4*b*), or steel X-bracing (fig 2-4*c*) but may be of wood in wood-frame buildings up to and including three stories. The shear wall concept for earthquake-resistant design of low buildings is quite valid. Its effectiveness depends primarily on the connections between the structural elements. Notable exceptions to the excellent performance of shear walls occur when the height-to-width ratio beco great enough to make overturning a problem when there are excessive openings in the shear

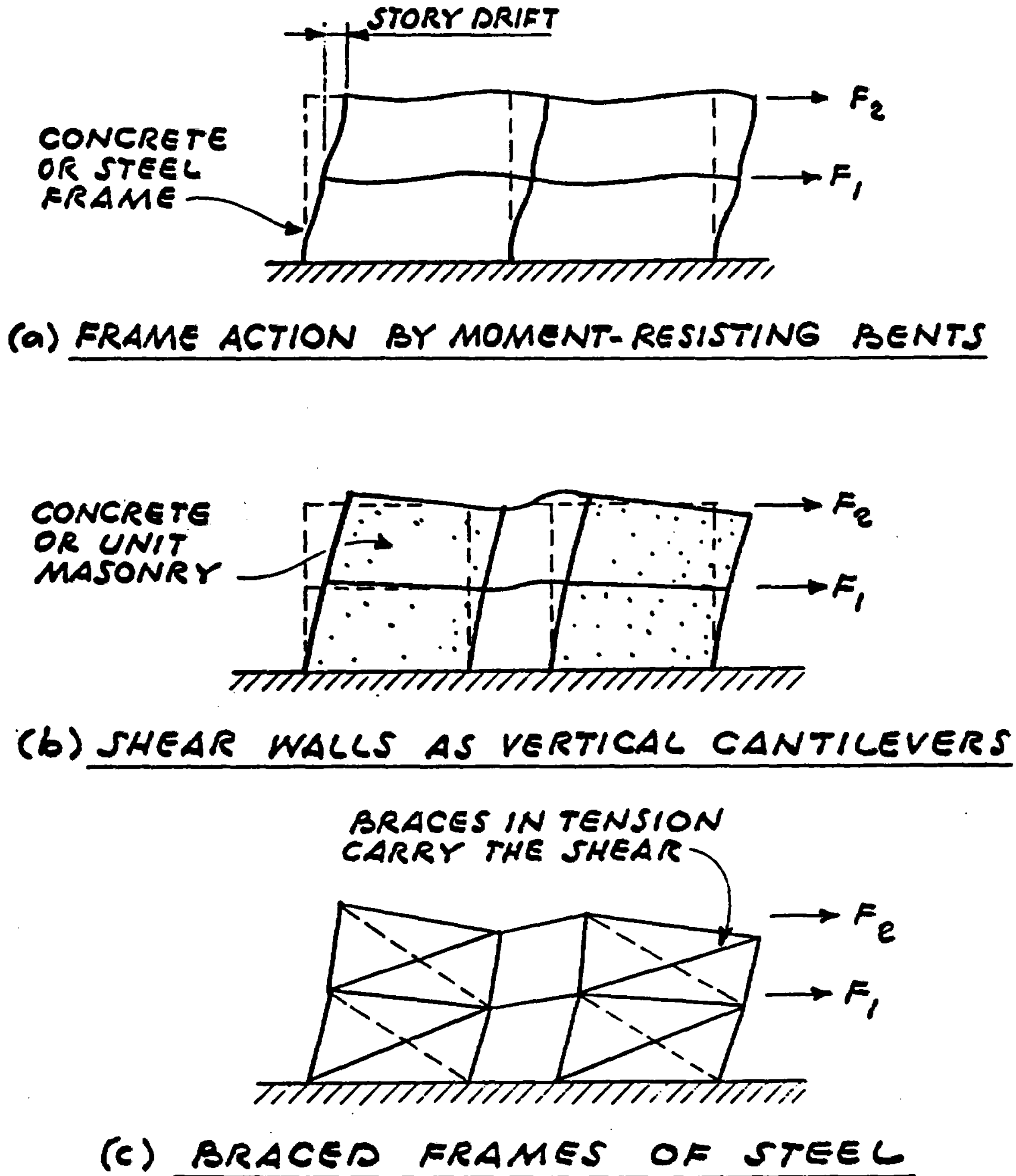

Figure 2-4. Basic lateral force-resisting systems

walls. Also, if the soil beneath its footings are relatively soft, the entire shear wall may rotate, causing localized damage around the wall.

(3) Either of the above structural systems may be used in combination with a wide variety of floor, roof, wall, and partition components. When frames and shear walls are combined, the system is called a dual bracing system. The type of structural system used, with specified details concerning the ductility and energy-absorbing capacity of its components, will establish the minimum K-value to be used for calculating the total base shear and to distribute the lateral seismic forces. The decision as to the type of structural system to be used shall be based on the merits and relative costs for the individual building being designed.

(4) The design engineer must be aware that a building does not merely consist of a summation of parts such as walls, columns, trusses, and similar components but is a completely integrated system or unit which has its own properties with respect to lateral force response. The designer must follow the forces through the structure into the ground and make sure that every connection along the path of stress is adequate to maintain the integrity of the system. It is necessary to visualize the response of the complete structure and to keep in mind that the real forces involved: are not static but dynamic; are usually erratically cyclic and repetitive; and can cause deformations well beyond those determined from the elastic design. Seismic forces are assumed to come from any horizontal direction and must be combined with gravity loads.

e. Diaphragms. Floor and roof systems are generally used as diaphragms. It is customary to design the floor and roof (e.g., concrete slab, wood sheathing, metal decking) as the web of a horizontal beam and to provide for the flange stresses of the beam with structural elements concentrated at the edge of the floor system (e.g., edge beams or special reinforcement in concrete slabs, continuous beams in wood and metal deck sytems). Too frequently, it is forgotten that these flanges must be made continuous or be adequately spliced. Horizontal truss systems may also be used as diaphragms (refer to chap 5, Diaphragms).

f. Shear Walls. The shear wall is designed as a vertical beam. To resist tensile stress due to bending moments, structural elements are concentrated at the vertical edges of walls in a manner similar to that described above for diaphragms. These boundary elements must be anchored into a foundation which is capable of transferring the forces into the ground (refer to chap 6, Walls).

g. Connections. Past performance of buildings earthquakes has shown that connections betwe floor and roof diaphragms and the shear walls are vulnerable to failure because of high stress concentrations. In order to develop the reserve capacity of the structural elements, the design forces for connections between lateral force-resisting elements are required to be greater than the design forces for the elements themselves (e.g., chap 3, para 3-3(J)1*g*, 3*a*, *b*, and *d*; and chap 4, para 4-6).

h. Ductility. Ductility is the capacity of building materials, systems, or structures to absorb energy by deforming in the inelastic range. The capability of a structure to absorb energy, with acceptable deformations and without failure, is a very desirable characteristic in any earthquake-resistant design. Structural steel (and wood to some degree) is considered to be a ductile material. Brittle materials such as concrete and unit-masonry must be properly reinforced with steel to provide the ductility characteristics necessary to resist seismic forces (chap 3, para 3-3(J)2b). In concrete columns, for example, the combined effect of flexure (due to frame action) and compression (due to the action of the overturning moment of the structure as a whole) produces a common mode of failure: buckling of the vertical steel and spalling of the concrete cover near the flr levels. Columns with proper spiral reinforcing hoops have a greater reserve strength and ductility (refer to chap 7, Space Frames).

i. Nonstructural Participation. For both analysis and detailing, the effects of nonstructural partitions, filler walls, and stairs (refer to chap 4, para 4-7*d*) must be considered. The nonstructural elements that are rigidly tied to the structural system can have a substantial influence on the magnitude and distribution of earthquake forces, causing a shearwall-like response with considerably higher lateral forces and overturning moments. Any element that is not strong enough to resist the forces that it attracts will be damaged; therefore, it should be isolated from the lateral force-resisting system.

j. Foundations. The differential movement of foundations due to seismic motions is an important cause of structural damage, especially in heavy, rigid structures that cannot accommodate these movements. Adequate design must minimize the possibility of relative displacement, both horizontal and vertical, between the various parts of the foundation and between the foundation and superstructure (refer to chap 3, para 3-3(J)3c; and chap 4, pa 4-8, for seismic requirements).

k. Damage Control Features. The design of a structure in accordance with the seismic provisions of this manual will not fully ensure against earthquake damage because the horizontal deformations from design loads are lower than those that can be expected during a major earthquake. However, without increasing construction costs, a number of things can be done to limit earthquake damage which would be expensive to repair. In considering a building's response to earthquake motions, it is important to keep in mind the structural system and the geometry of the building. During a major earthquake it should be assumed that deflections (story drift) may be 3/K times that resulting from the design lateral forces (refer to chap 3, para 3-3(J)1*d*). A list of features to minimize damage follows:

(1) Provide details which allow structural movement without damage to nonstructural elements. Damage to such items as piping, glass, plaster, veneer, and partitions may constitute a major financial loss. To minimize this type of damage, special care in detailing, either to isolate these elements or to accommodate the movement, is required.

(2) Breakage of glass windows can be minimized by providing adequate clearance and flexible mountings at edges to allow for frame distortions.

(3) Damage to rigid nonstructural partitions can be largely eliminated by providing a detail at the top and sides which will permit relative movement between the partitions and the adjacent structural elements.

(4) In piping installations, the expansion loops and flexible joints used to accommodate temperature movement are often adaptable to handling the relative seismic deflections between adjacent equipment items attached to floors.

(5) Fasten free-standing shelving to walls to prevent toppling.

(6) Concrete stairways often suffer seismic damage due to their inhibition of drift between connected floors. This can be avoided by providing a slip joint at the lower end of each stairway to eliminate the bracing effect of the stairway or by tying stairways to stairway shear walls.

l. Redundancy. Redundancy is a highly desirable characteristic for earthquake-resistant design. When the primary element or system yields or fails, the lateral force can be redistributed to a secondary system to prevent progressive failure.

2-10. Alternatives to the prescribed provisions. Alternatives to some of the seismic provisions are permitted if they can be properly substantiated. In some cases, alternative solutions are mandatory (e.g., irregular buildings and setback buildings); in other cases, they are optional (e.g., to provide a more efficient design or to analyze the building for the effects of a predicted earthquake ground motion). The alternatives are generally classified as dynamic methods and are not covered in this manual. Using dynamic loading and a computer analysis, one can more accurately predict how a proposed building will act and deform under ground motions from a specific earthquake. It will be found that this response may sometimes cause deflections, joint rotations, and stresses quite different from those determined from the prescribed static loadings. Before proceeding with the equivalent static force procedure, the designer should make sure that there are no special conditions that would warrant or require the use of more rigorous methods.

a. Elastic Analysis. For most buildings requiring an alternative design method, an elastic dynamic analysis procedure is sufficient to determine load distribution and member forces for design earthquake motion. A response spectrum analysis with the modes combined by the square-root-of-the-sum-of-the-squares (SRSS) method or by some other approved method is generally sufficient for an elastic analysis. A time-history analysis may be used if necessary.

b. Inelastic Analysis. For major buildings, which require added assurance so that the building can withstand a major earthquake without collapse or within a limited range of damage, an inelastic dynamic analysis may be used. This usually is a time-history analysis; however, other approximate procedures that can estimate inelastic effects may be used.

2-11. Future expansion. When future expansion of a building is contemplated, it is generally better to plan for horizontal expansions rather than for vertical growth because there will be greater freedom in planning the future increment, there will be less interruption of existing operations when additions are made, and the first increment will not have to bear a large share of cost of the second increment. For future vertical expansion, the foundation, floor/roof system, and the structural frame must be proportioned for both the initial and future design loadings, including the seismic forces. For future horizontal expansion, either a complete structural separation between the two phases must be provided, or the first increment must be designed for its share of the loads under both conditions: the first increment and the expansion. Many buildings that have been designed for future expansion under past seismic criteria do not satisfy the present criteria;

therefore, these buildings must be upgraded and will incur high seismic strengthening costs.

2-12. Major checkpoints. The process of achieving an adequate building must start with conceptual planning and be carried through all phases of the design and construction program. The major check points include: perform site investigations; coordinate the work of the architect and engineers (structural, mechanical, and electrical) to establish the plan, the system, and the materials of construction; establish design criteria for the specific facility; identify and locate primary structural elements; determine and distribute lateral seismic forces; prepare design calculations; detail connections; detail nonstructural parts for damage control; make clear, complete contract drawings; check shop drawings; perform quality control inspection; and maintain surveillance over any changed conditions during the entire construction period.

CHAPTER 3
DESIGN CRITERIA

3-1. Purpose and scope. This chapter prescribes the criteria for the seismic design of buildings and other structures.

3-2. General. The seismic design of buildings and other structures will be in accordance with the criteria and design standards herein. The structural system or type of construction will admit to a rational analysis in accordance with established principles of mechanics. Structures will be designed for dead, live, snow, wind, and seismic forces. The dead, live, snow, and wind loads will be as given in applicable agency manuals. Every building or structure and every portion thereof will be designed and constructed to resist stresses produced by lateral seismic forces in combination with dead and live loads as provided in this chapter. Materials and details will conform to the seismic provisions, applicable guide specifications, and criteria herein. The provisions of this chapter apply tc the structure as a unit and also to all parts thereof, including the structural frame or walls, floor and roof systems, anchorages and supports for architectural elements and mechanical and electrical equipment, and other elements.

3-3. Seismic design provisions. The seismic provisions of this manual are based on the "Recommended Lateral Force Requirements and Commentary" of the Seismology Committee of the Structural Engineers Association of California, Fourth Edition, 1975[1] (hereafter referred to as the SEAOC Recommendations). The SEAOC publication, which includes the Recommendations, the Commentary, and Appendices, may be used as a reference for this manual. (Note: The SEAOC Commentary discusses and explains the provisions of the SEAOC Recommendations Lateral Force Requirements. In some respects, the Commentary is as important as the Recommendations. The Commentary, in general, gives the intent of the seismic provisions; however, it becomes an extension of the SEAOC Recommendations when supplementing the seismic provisions with clarifying interpretations.) The following is a reprinted version of Section 1 of the SEAOC Recommendations that has been modified to satisfy the requirements of this manual (see chap 6 and 7 for references to SEAOC Sections 2, 3, and 4). The modifications consist primarily of (1) additions and interpretations which extend the provisions to more fully cover areas of lower seismicity, outside of California, (2) special provisions developed by the Tri-Services Committee; and (3) 1978 SEAOC Seismology Committee revisions. Modified portions are noted in *italics*. The SEAOC paragraph identification system has been maintained such that SEAOC Section 1(J)2d is equivalent to paragraph 3-3(J)2d in this manual.

[1]Published by the Structural Engineers Association of California, 171 Second Street, San Francisco, California 94105.

SEAOC, SECTION 1*

GENERAL REQUIREMENTS FOR THE DESIGN AND CONSTRUCTION OF EARTHQUAKE RESISTIVE STRUCTURES
(Modifications are in Italics)

(A) General.

1. 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.

2. 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.

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

4. The effects of vertical accelerations must be considered for structures in Seismic Zones 3 and 4 (chap 4, para 4-4c(2)).

5. Dead, live, snow, and wind loads will be in accordance with applicable agency manuals. Earthquake loads will be considered in combination with dead loads and live loads as specified in paragraph 3-3(J)2c. Allowable working stresses specified in agency manuals for ordinary or non-seismic construction will be increased one-third for earthquake loading, provided the required section or area computed on this basis is not less than that required for vertical loading, without the one-third increase. Working stresses for reinforced masonry construction will be as given in chapter 8, Reinforced Masonry. The one-third increase in stresses does not apply when strength design or plastic design methods are used.

(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 sytem, the required lateral forces are resisted by shear walls or braced frames as hereinafter defined. *Refer to chapter 4, paragraph 4-3c(4).*

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. *Refer to chapter 6.*

DUCTILE MOMENT RESISTING SPACE FRAME is a moment resisting space frame *that complies with special requirements given in chapter 7. To comply with the SEAOC Recommendations, only Type A concrete and steel frames could be classified as ductile moment resisting space frames; however, in this manual the definition is extended to include concrete frame Type B for buildings in Seismic Zone 1.*

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 *paragraph 3-3(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. *Classifications are given in chapter 7.*

SHEAR WALL is a wall designed to resist lateral forces parallel to the plane of the wall. *Classifications are given in chapter 6.*

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. *Refer to chapter 4, paragraph 4-3c(4).*

(C) Symbols and Notations.

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

- C = Numerical coefficient as specified in *paragraph 3-3(D)*
- C_p = Numerical coefficient as specified in *paragraph 3-3(G) and as set forth in table 3-4.*
- D = The dimension of the building in feet, in a direction parallel to the applied forces.
- δ_i = *Deflection at level i relative to the base, due to applied lateral forces, Σf_i, for use in Formula (3-3).**
- 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.
- f_i = *Distributed portion of a total lateral force at level i for use in formula (3-3).**
- 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 3-3.
- 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_{px} = *The weight of the diaphragm and the elements tributary thereto at level x, including 25 percent of the floor live load in storage and warehouse occupancies.**
- 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 *paragraphs 3-3(G) and 3-3(I)*, every structure shall be designed and constructed to resist minimum total lateral seismic forces as-

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sumed to act nonconcurrently in the direction of each of the main axes *(chap 4-4c(1))* of the structure in accordance with the formula

$$V = ZIKCSW. \tag{3-1}$$

However, the product of ZIKCS will not be less than 0.015.

1. The value of Z is *dependent upon the seismic zone as specified by figures 3-1, 3-2, 3-3, and 3-4 in paragraph 3-4 and is determined from table 3-1 below.*

Table 3-1. Z-Coefficient

Seismic Zone	0	1	2	3	4
Z-coefficient	0	3/16	3/8	3/4	1

2. The value of the coefficient I *is dependent on the type occupancy, such as discussed in paragraph 3-5, and is determined from table 3-2 below:*

Table 3-2. I-Coefficient

Type of Occupancy	*I*
Essential Facilities	1.50
High Risk Facilities	1.25
All Others	1.00

3. The value of K shall be not less than that set forth in table 3-3.

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

5. W is the total dead load and applicable portions of other loads *including all permanent structural and nonstructural components of a building such as walls, floors, roofs, and fixed service equipment.*

a. Where partition locations are subject to change, in addition to all other loads, a uniformly distributed dead load of 20 pounds per square foot of floor will be applicable.

b. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load will be applicable.

c. Where the design uniform snow load is 20 psf or less, no part need be included in the value of "W". Where the snow load is greater than 20 psf, an effective weight of 70 percent of the full snow load will be included; however, where the snow load duration warrants, the effective weight of the snow load may be reduced to 20 percent of the full snow load.

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

$$C = \frac{1}{15\sqrt{T}} \tag{3-2}$$

The value of C need not exceed 0.12.

7. 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

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div \left(g \sum_{i=1}^{n} f_i\delta_i\right)} \qquad (3\text{-}3)^*$$

where the values of f_i represent any lateral force distributed approximately in

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accordance with the principles of formulas 3-5), (3-6), and (3-7) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.* *(Refer to chap 4, para 4-3d.)*

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

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

or, for buildings in which the lateral force resisting system consists of moment resisting space frames capable of resisting 100 percent 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

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

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

$$\text{For } \frac{T}{T_s} = 1.0 \text{ or less, } S = 1.0 + \frac{T}{T_s} - 0.5\left[\frac{T}{T_s}\right]^2. \qquad (3\text{-}4)$$

$$\text{For } \frac{T}{T_s} \text{ greater than } 1.0, S = 1.2 + 0.6\frac{T}{T_s} - 0.3\left[\frac{T}{T_s}\right]^2. \qquad (3\text{-}4A)$$

T in Formulas (3-4) and (3-4A) shall be established by a properly substantiated analysis but T shall not be taken as less than 0.3 seconds

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 seconds nor more than 2.5 seconds. 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 seconds, the value of S may be determined by assuming a value of 2.5 seconds 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:

$$V=F_t+\sum_{i=1}^{n} F_i. \qquad (3\text{-}5)$$

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

$$F_t=0.07\,TV. \qquad (3\text{-}6)$$

F_t need not exceed 0.25V and may be considered as zero where T is 0.7 seconds 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

$$F_x=\frac{(V-F_t)\,w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (3\text{-}7)$$

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 percent of the corresponding plan dimen-

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sion 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. The forces shall not be decreased due to torsional effects. 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 five percent 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 to the main structural system shall be designed for lateral forces in accordance with the formula**

$$F_p = ZIC_pW_p^{*} \qquad (3\text{-}8)$$

The distribution of these forces shall be according to the gravity loads pertaining thereto.

1. *The values of C_p are set forth in table 3-4. The value of the I coefficient shall be the value used for the building.**

EXCEPTIONS:

*a. The value of I for wall panel connectors shall be as given in paragraph 3-3(J)3d.**

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*b. The value of I for elements of life safety systems (such as items associated with exiting and fire protection) shall be 1.5.**

*2. For applicable forces on diaphragms and connections for exterior panels, refer to paragraphs 3-3(J)2d and 3-3(J)3d, respectively.**

3. For applicable forces on flexible and flexibly mounted equipment and machinery (footnote 3, table 3-4), refer to chapter 10 (equipment in buildings).

4. For applicable forces on storage racks, refer to chapter 9 (footnote 5, table 3-4).

5. For applicable forces on lighting fixtures, piping, stacks, bridge cranes and monorails, and elevators, refer to chapter 10.

(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. *Refer to chapter 4, paragraph 4-7.*

(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. *(Some exceptions are permitted for dual systems with height limitations as specified in table 3-7.)*

b. Tall Buildings. *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 percent of the required seismic forces for the structure as a whole.

> *EXCEPTION: Buildings more than 160 feet in height in Seismic Zone No. 1 may have concrete shear walls designed in conformance with chapter 6, paragraph 6-3a(1), in lieu of a ductile moment resisting space frame, providing a K value of 1.00 or 1.33 is utilized in design.*

c. Concrete Frames. *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. *(Some exceptions are permitted in Seismic Zones No. 1 and No. 2 with height limitations as specified in table 3-7.)*

> *EXCEPTION:* Frames in the perimeter line of vertical support of buildings designed with shear walls taking 100 percent of the design lateral forces need only conform with *paragraph 3-3(J)1d.*

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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 *paragraph 3-3(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 *structural steel or reinforced concrete frame complying with the requirements of chapter 7 and conforming to the classifications of tables 3-3 and 3-7.*

g. Braced Frames. All members in braced frames shall be designed for 1.25 times the force determined in accordance with *paragraph 3-3(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 *comply with the requirements of chapter 6, paragraph 6-7, and conform to the classifications of tables 3-3 and 3-7.*

h. Shear Walls. Reinforced concrete shear walls for all structures shall conform to the requirements of *chapter 6, paragraph 6-3, and conform to the classifications of tables 3-3 and 3-7. Reinforced masonry shear walls shall conform to the requirements of chapter 8.* 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 *paragraph 3-3(D).*

i. Framing Below Base. In buildings where K = 0.67 or 0.80, the special ductility requirements *of SEAOC sections 2 (chapter 7, paragraph 7-3a(1)), 3 (chapter 6, paragraph 6-3a(1)), and 4 (chapter 7, paragraph 7-5a(1)),* 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. *All* elements within the structure which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete *under the provisions of chapters 6 and 8.*

EXCEPTION: See table 8-5 for Seismic Zone 1 exceptions.

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 *and collectors shall be designed to resist the seismic forces determined in accordance with the following formula:*

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$$F_{px} = \left(\frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} \right) w_{px} \qquad (3\text{-}9)$$

The force F_{px} determined from Formula 3-9 need not exceed 0.30 Z I w_{px}.
When the diaphragm is required to transfer seismic forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offsets in the placement of the elements or to changes in the stiffness in the vertical elements these forces shall be added to those determined from Formula 3-9.

However, in no case shall the seismic force on the diaphragm be less than determined by the following formula:

$$F_{px} = 0.14\, Z\, I\, w_{px} \qquad (3\text{-}9A)$$

Diaphragms supporting concrete or masonry walls shall have continuous ties between diaphragm chords to distribute the anchorage forces specified in *paragraph 3-3(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 paragraph 3-3(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 *paragraph 3-3(J)2d* for the requirements for developing anchorage forces in diaphragms. See *paragraph 3-3(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 *paragraph 3-3(J)3a. In Seismic Zones No. 2, No. 3, and No. 4* 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 *paragraph 3-3(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. *See chapter 4, paragraph 4-8, for supplemental requirements.*

d. Exterior Elements. Precast or prefabricated nonbearing, nonshear wall panels or similar elements which are attached to or enclose the exterior, shall*

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be designed to resist the forces per Formula 3-8 and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Concrete panels or other similar elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

(1) *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/K) times the calculated elastic story displacement caused by required seismic forces, or 1/2-inch, whichever is greater.*

(2) *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 or other connections providing equivalent sliding and ductility capacity.*

(3) *Bodies of connections, such as structural steel angles, rods, plates, etc., shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete or brittle failures at or near welds. The body of the connection shall be designed for 1.33 times the force determined by Formula 3-8.*

(4) *Elements connecting the body to the panels or the structure, such as bolts, inserts, welds, dowels, etc., shall be designed for 4 times the forces determined by Formula 3-8. Elements of connections embedded in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to effectively transfer forces to the reinforcing steel.*

(5) *The value of the coefficient I in Formula 3-8 shall be 1.0 for the entire connection (i.e., the value need not be greater than 1.0 even if the I-coefficient of the building is greater than 1.0).*

e. **Connections.** *For additional requirements for connections refer to chapter 4, paragraph 4-6.*

*Table 3-3. Horizontal Force Factor "K" for Buildings or Other Structures**
(Refer to Table 3-7 (Paragraph 3-6) for Summary Tables for K Values for Each Seismic Zone.)

Basic System	Category	Type or Arrangement of Resisting Elements	Value of K[a,b]
100% Frames	1	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
	2	Buildings with moment resisting space frames designed in accordance with the following criteria: The moment resisting space frame shall have the capacity to resist the total required lateral force and shall comply with the height limitations and frame specifications of Table 3-7.	1.00
Dual Systems	3	Buildings with a dual bracing system consisting of a moment resisting space frame and shear walls or braced frames designed in accordance with the following criteria: a. The moment resisting space frames shall comply with the specifications and height limitations of Table 3-7. b. 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. c. The shear walls or braced frames acting independently of the moment resisting space frame shall resist the total required lateral force. d. The moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force.	0.80
100% Walls or Braced Frames	4	Buildings with a vertical load carrying space frame and shear walls or braced frames designed in accordance with the following criteria: a. In Seismic Zones 2, 3, and 4 the height of the building shall not exceed 160 feet.[c] b. The shear wall or braced frame shall have the capacity to resist the total required lateral force and shall comply with the height limitations and wall specifications of Table 3-7. c. The interaction between the vertical load carrying space frame and the shear walls or braced frames shall not result in the loss of the vertical load carrying capacity of the space frame in the case of damage occurring to a portion of the lateral force resisting system (see paragraph 3-3(J)1d).	1.00
	5	Building with wood frame construction and plywood shear walls designed in accordance with the following criteria:** a. The height of the building shall not exceed 40 feet or three stories. b. The plywood shear walls shall have the capacity to resist the total required lateral force. c. Masonry veneers shall not be used. (If veneers are used, K = 1.33.)	1.00
	6	Buildings with a box system designed in accordance with the following criteria: a. In Seismic Zones 2, 3, and 4 the height of the building shall not exceed 160 feet.[c] b. The shear walls or braced frames shall have the capacity to resist the total lateral force and shall comply with the height limitations and wall specifications of Table 3-7.	1.33[a]
Elevated Tanks and Inverted Pendulums	7	Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building. The braced frame requirements of paragraph 3-3(J)1g and the torsional requirements of paragraph 3-3(E)5 shall apply. The product of KCS will not be less than 0.12. Refer to Chapter 11 for inverted pendulums.[d]	2.5[d]
Structures Other Than Buildings	8	Structures other than buildings, elevated tanks, or minor structures set forth in Table 3-4. The product of KCS will not be less than 0.10. Also, refer to Chapter 11.[d]	2.0

*Modification of SEAOC Table 1A.

**In 1980 SEAOC modified this category to include "buildings--with stud wall framing and using horizontal diaphragms and vertical shear panels for the lateral force system." Therefore, walls in accordance with either paragraph 6-5a or paragraph 6-5b of Chapter 6 will be in compliance with item 5b above.

Footnotes to Table 3-3

a. If K = 1.33 in one direction, it will be 1.33 in both directions. Other K-values may vary in the two directions.
b. Generally, one value of K applies to the total height of the building; however, if there is a change in K along the height of the building (e.g., due to change in framing system), the K value used at any level must be equal to or greater than the K value at the next level above. (Also, refer to provisions of paragraphs 3-3(E)1 and 2 for setback and irregular buildings.)
c. In Seismic Zone 1 concrete shear walls may exceed the 160-foot limit (paragraph 3-3(J)1b).
d. Categories 7 and 8: Refer to chapter 11 for alternate methods and additional requirements. Pedestal type elevated water tanks will not be permitted in Seismic Zone Nos. 3 and 4. In Seismic Zone Nos. 1 and 2, K will be 3.0 for pedestal type elevated tanks.

*Table 3-4. Horizontal Force Factor "C_p" for Elements of Structures**

	Part or Portion of Structure	Horizontal Direction of Force	Value of C_p[1]
1	Cantilever Elements: a. Parapets b. Portion of chimneys or stacks that protrude above rigid supports[2]	Normal to flat surfaces Any direction	0.8
2	All other elements such as walls, partitions and similar elements—see also paragraph 3-3(J)3d. Also includes masonry or concrete fences over 6 feet high.	Any direction	0.3
3	Exterior and interior ornamentations and appendages. See chapter 9, paragraph 9-3.	Any direction	0.8
4	When connected to, part of, or housed within a building: a. Penthouses b. Anchorage and supports for tanks plus contents c. Rigidly braced chimneys and stacks[2] d. Storage racks plus contents[5] e. Suspended ceilings[6] f. All equipment or machinery	Any direction	0.3[3,4]
5	Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any direction	0.3[4]

*Based on the 1978 SEAOC Revisions

Footnotes to Table 3-4

1. *C_p for elements laterally self supported only at ground level may be 2/3 of the value shown. Also refer to chapters 10 and 11 (e.g., equipment, paragraph 10-5; stacks, paragraph 10-8, and tanks, chapter 11).*
2. *Chimneys or stacks that extend more than 15 feet above a rigid attachment to the structure will be designed in accordance with chapter 10, paragraph 10-8a. Also, refer to chapter 10 for guyed stacks and stacks on ground.*
3. *For flexible and flexibly mounted equipment and machinery, the appropriate values of 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 but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery (refer to chapter 10).

For Essential Facilities and life safety systems (i.e., when the value of the I-coefficient is equal to 1.5 per paragraph 3-3(G)1), the design and detailing of equipment which must remain in place and be functional following a major earthquake shall consider the effect of drift.

4. *The force shall be resisted by positive anchorage and not by friction.*
5. *W_p for storage racks shall be the weight of the racks plus contents. The value of C_p for racks over two storage support levels in height shall be 0.24 for the levels below the top two levels. In lieu of the tabulated values, steel storage racks may be designed in accordance with chapter 9, paragraph 9-4g.*
6. *Ceiling weight shall include all light fixtures and other equipment or partitions which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used.*

3-4. Seismic zone maps. The seismic zones required for the determination of the coefficient Z in table 3-1, paragraph 3-3(D)1, are given on maps shown on figures 3-1, 3-2, and 3-3 for the contiguous states, Alaska, and Hawaii, respectively. The map on figure 3-4 shows the seismic zones for California and Nevada in greater detail and scale. Seismic zones for specific areas are tabulated in tables 3-5 and 3-6 for localities within the United States and outside the United States, respectively. The boundary lines are approximate, and in the event of any conflict or uncertainty regarding the applicable zone of any particular site, the higher zone will be used.

3-5. Types of occupancy. General descriptions and examples of various occupancy types are given for the determination of the value of the coefficient I in table 3-2, paragraph 3-3(D)2.

a. Essential Facilities (I = 1.5). These are structures housing critical facilities which are necessary for post-disaster recovery and require continuous operation during and after an earthquake. This includes facilities where damage from an earthquake may cause significant loss of strategic and general communications and disaster response capability. Typical examples are:

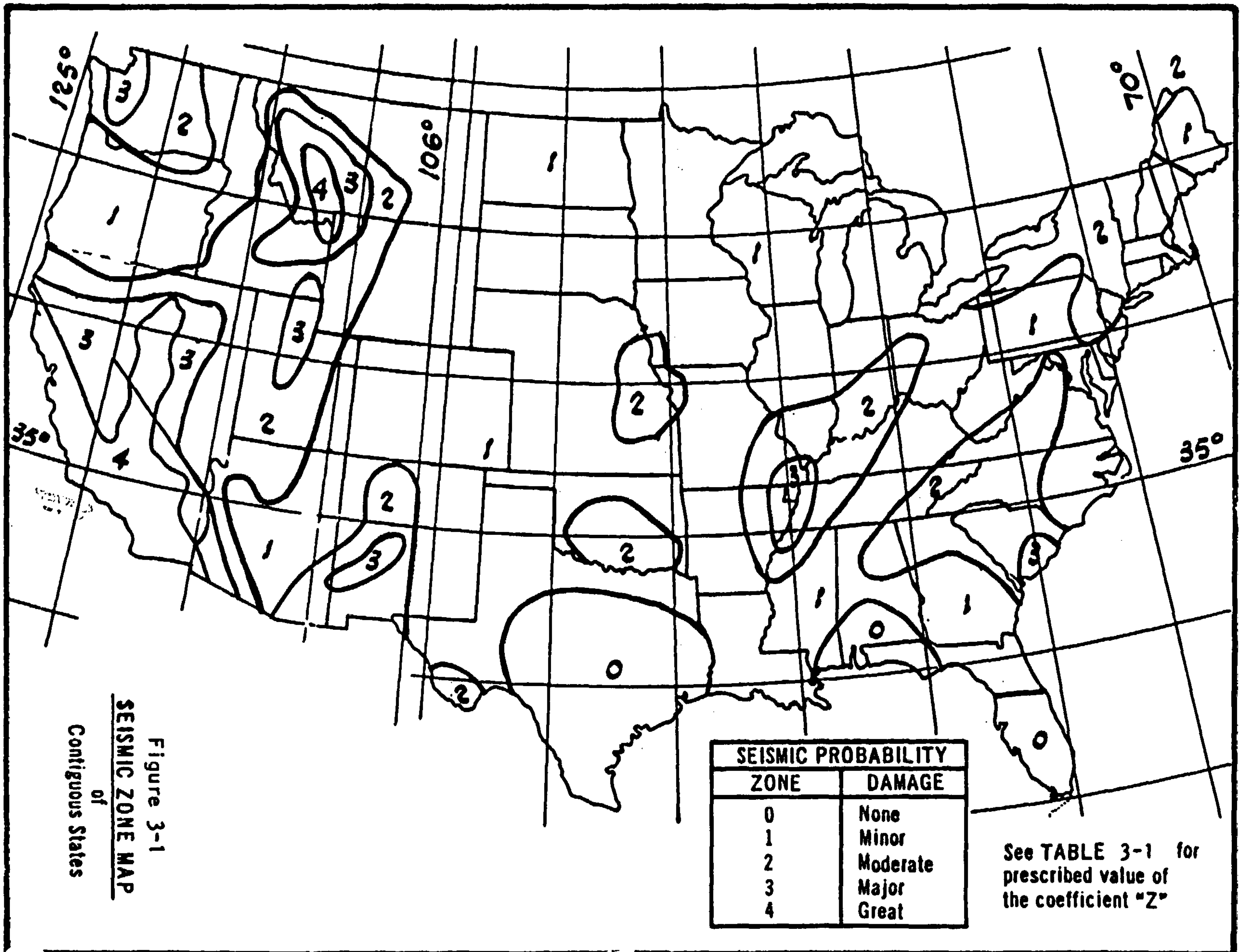


Figure 3-1
SEISMIC ZONE MAP
of
Contiguous States

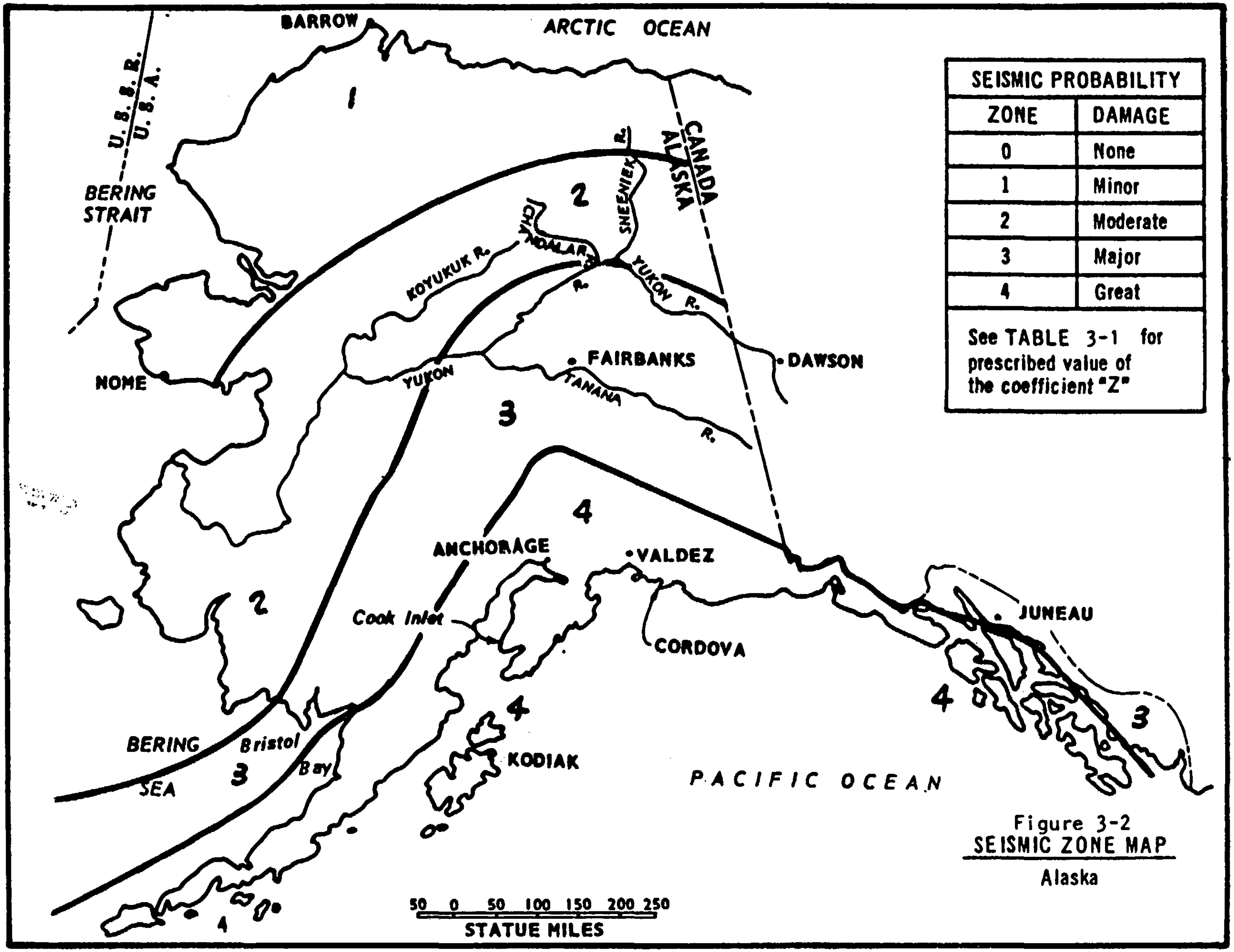
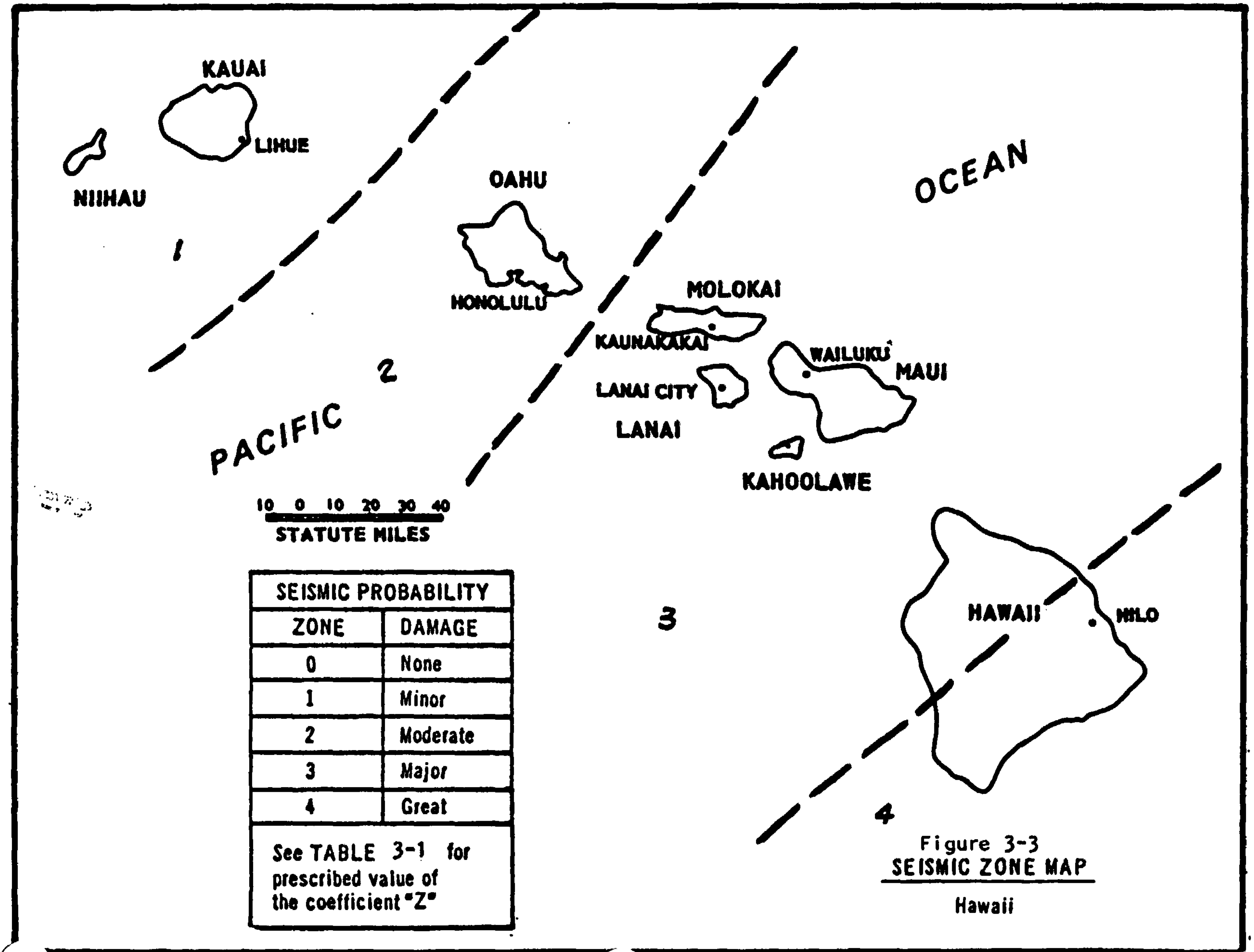


Figure 3-2
SEISMIC ZONE MAP
Alaska



SEISMIC PROBABILITY	
ZONE	DAMAGE
0	None
1	Minor
2	Moderate
3	Major
4	Great

See TABLE 3-1 for prescribed value of the coefficient "Z"

Figure 3-3
SEISMIC ZONE MAP
Hawaii

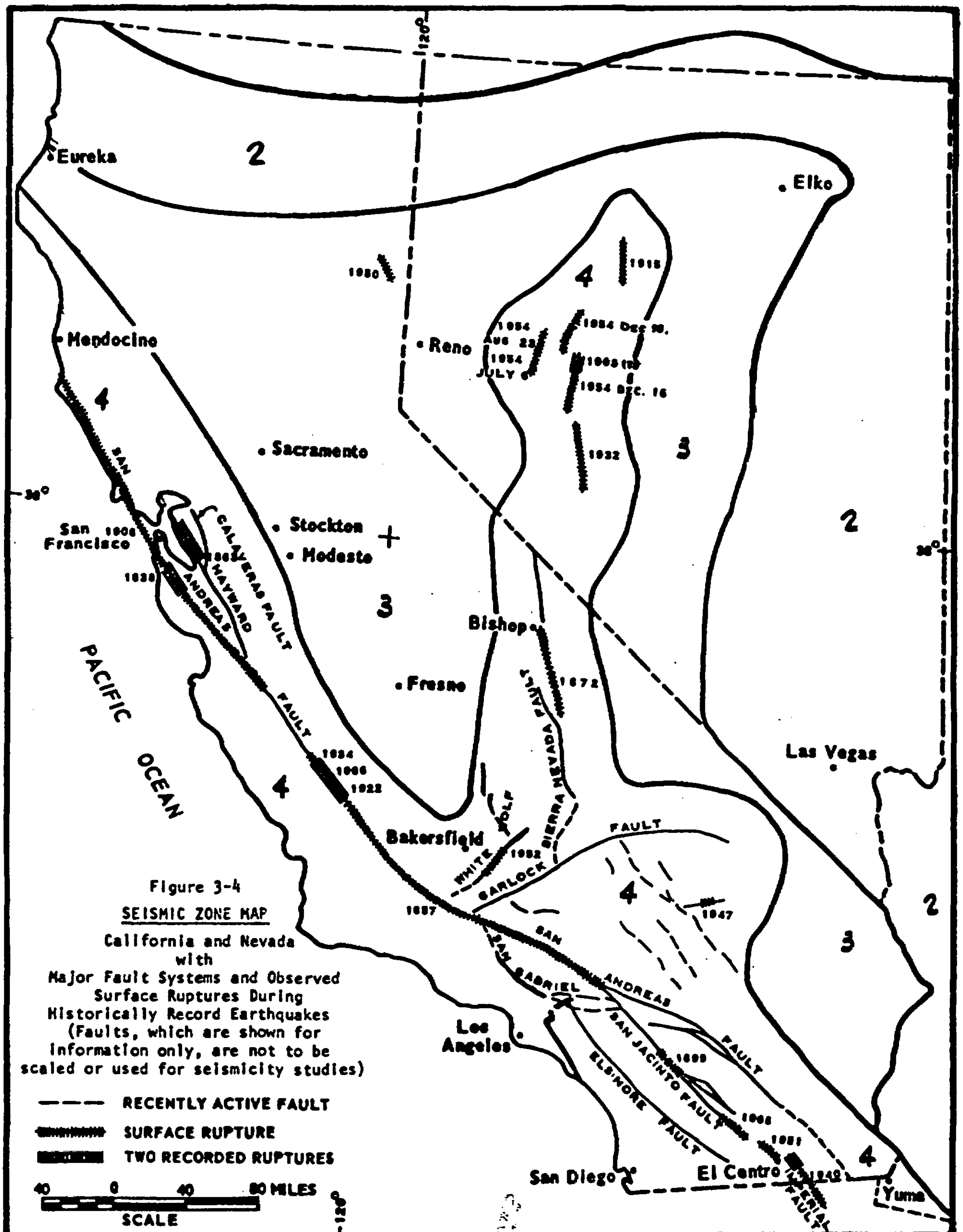


Figure 3-4

SEISMIC ZONE MAP

California and Nevada
with
Major Fault Systems and Observed
Surface Ruptures During
Historically Record Earthquakes
(Faults, which are shown for
information only, are not to be
scaled or used for seismicity studies)

— — — RECENTLY ACTIVE FAULT
SURFACE RUPTURE
TWO RECORDED RUPTURES

40 0 40 80 MILES
SCALE

*Table 3-5. Seismic Zone Tabulation, U.S.**

ALABAMA	
Anniston	2
Maxwell AFB	0
Birmingham	2
Huntsville	1
Mobile	0
Montgomery	0
Fort Rucker	0
ALASKA	
Adak Island	4
Anchorage	4
Barrow	1
Bethel	2
Eielson AFB	3
Elmendorff AFB	4
Fairbanks	3
Fort Greely	3
Juneau	3
Kodiak Island	4
Nome	1
ARIZONA	
Fort Huachuca	2
Luke AFB	1
Navajo AD	1
Phoenix	1
Tucson	1
Williams AFB	1
Yuma	4
ARKANSAS	
Blytheville AFB	3
Fort Chaffee	1
Little Rock AFB	1
CALIFORNIA	
Castle AFB	3
China Lake	4
Edwards AFB	4
Hamilton AFB	4
Hunter-Ligget MR	4
Long Beach	4
Los Angeles	4
March AFB	4
Mare Island	4
Norton AFB	4
Oakland	4
Fort Ord	4
Camp Pendleton	4
Port Hueneme	4
Sacramento	3
San Diego	4
San Francisco	4
Sharpe AD	3
Sierra AD	3
Travis AFB	4
Vandenberg AFB	4
COLORADO	
USAF Academy	1
Fort Carson	1
Denver	1
Fitzsimons AMC	1
Peterson Field	1
Pueblo	1
CONNECTICUT	
Hartford	2
New Haven	2
New London	2
DELAWARE	
Dover AFB	1
Wilmington	2
FLORIDA	
Eglin AFB	0
Homestead AFB	0
Jacksonville	1
Key West	0
MacDill AFB	0
Miami	0
Orlando	0
Patrick AFB	0
Pensacola	0
Tampa	0
Tyndall AFB	0
GEORGIA	
Albany	1
Atlanta	2
Fort Benning	1
Fort Gordon	2
Hunter AFB	2
Macon	1
Robbins AFB	1
Savannah	2
Fort Stewart	1
HAWAII	
Barbers Point, Oahu	2
Hickam AFB	2
Hilo, Hawaii	4
Honolulu, Oahu	2
Kaneohe Bay, Oahu	2
Lihue, Kauai	1
Schofield Barracks	2
Wheeler AFB	2
IDAHO	
Idaho Falls	2
Mountain Home AFB	1
ILLINOIS	
Chanute AFB	1
Chicago	1
Great Lakes TC	1
Joliet AAP	1
O'Hare IAP	1
Rock Island Arsenal	1
Savanna AD	1
Scott AFB	2
INDIANA	
Fort Ben Harrison	2
Fort Wayne	2
Grissom AFB	1
Indiana AAP	2
IOWA	
Burlington	1
Cedar Rapids	1
Des Moines	1
Sioux City	1
KANSAS	
Kansas AAP	1
Fort Leavenworth	2
McConnell AFB	1
Fort Riley	2
Sunflower AAP	2
KENTUCKY	
Fort Campbell	[illegible]
Lexington	[illegible]
Louisville	1
Fort Knox	2
LOUISIANA	
Fort Polk	1
Lake Charles	1
Louisiana AAP	1
New Orleans	1
Shreveport	1
MAINE	
Bangor	1
Brunswick	2
Loring AFB	1
Winter Harbor	1
MARYLAND	
Aberdeen Proving Ground	1
Andrews AFB	1
Annapolis	1
Baltimore	1
Fort Detrick	1
Edgewood Arsenal	1
Fort Meade	1
Fort Ritchie	1

*Refer to table 3-1 for prescribed values of Z.

*Table 3-5. Seismic Zone Tabulation, U.S.**

Location	Zone
MASSACHUSETTS	
Boston	2
Fort Devens	2
L. G. Hanscom Field	2
Otis AFB	2
Westover AFB	2
MICHIGAN	
Detroit	1
Kincheloe AFB	1
K. I. Sawyer AFB	1
Selfridge AFB	1
Wurtsmith AFB	1
MINNESOTA	
Duluth	1
Minneapolis	1
Osceola AFB	1
MISSISSIPPI	
Biloxi	0
Columbus AFB	1
Jackson	1
Keesler AFB	0
Meridan	1
MISSOURI	
Kansas City	2
Lake City AAP	2
Fort Leonard Wood	1
St. Louis	2
Richards Gebaur AFB	2
Whiteman AFB	1
MONTANA	
Helena	3
Malmstrom AFB	2
Missoula	2
NEBRASKA	
Cornhusker AAP	1
Lincoln	1
Offutt AFB	1
NEVADA	
Carson City	3
Fallon	4
Hawthorne	4
Las Vegas	2
NEW HAMPSHIRE	
Hanover	2
Pease AFB	2
Portsmouth	2
NEW JERSEY	
Atlantic City	1
Bayonne	2
Picatinny Arsenal	2
McGuire AFB	1
Fort Monmouth	2
NEW YORK	
Albany	2
Buffalo	2
Fort Drum	2
Griffiss AFB	2
New York	2
Niagara Falls IAP	2
Plattsburg AFB	2
Syracuse	1
West Point Military Reservation	2
Watervliet	2
NORTH CAROLINA	
Fort Bragg	1
Charlotte	2
Camp Lejeune	1
Greensboro	2
Pope AFB	1
Seymour Johnson	1
Sunny Point Ocean Terminal	1
NORTH DAKOTA	
Bismarck	1
Fargo	1
Grand Forks AFB	1
Minot AFB	1
OHIO	
Cincinnati	1
Cleveland	1
Columbus	1
Ravenna AAP	1
Wright-Patterson AFB	1
OKLAHOMA	
Enid/Vance AFB	1
Fort Sill	2
Tinker AFB	2
Tulsa	1
OREGON	
Coos Bay	1
Eugene	1
Portland	1
Umatilla AD	1
PENNSYLVANIA	
Carlisle Barracks	1
Harrisburg	1
Letterkenny AD	1
Philadelphia	2
Pittsburgh	1
Scranton	2
NEW MEXICO	
Albuquerque	2
Cannon AFB	1
Hollomon AFB	2
White Sands MR	2
RHODE ISLAND	
Newport	2
Providence	2
SOUTH CAROLINA	
Charleston	3
Fort Jackson	2
Parris Island	3
Shaw AFB	2
SOUTH DAKOTA	
Ellsworth AFB	1
Pierre	1
Sioux Falls	1
TENNESSEE	
Chattanooga	2
Holston AAP	2
Memphis	3
Milan AAP	3
Nashville	1
TEXAS	
Austin/Bergstrom AFB	0
Corpus Christi	0
Dallas	0
Dyess AFB	0
Ellington AFB	0
El Paso	2
Galveston	0
Fort Hood	0
Houston	0
Lone Star AAP	1
Reese AFB	1
San Antonio	0
Fort Worth	0
Wichita Falls	~~0~~
UTAH	
Dugway P.G.	2
Hill AFB	3
Salt Lake City	3
Tooele Army Depot	3
VERMONT	
All	2
VIRGINIA	
Fort Belvoir	1
Fort Eustis	1
Fort Meyer	1
Norfolk	1
Petersburg/Fort Lee	1
Quantico	1
Radford AAP	2
Richmond	1

*Refer to table 3-1 for prescribed values of Z.

*Table 3-5. Seismic Zone Tabulation, U.S.**

WASHINGTON	
Bremerton	3
Fairchild AFB	1
Fort Lewis	3
McChord AFB	3
Seattle	3
Walla Walla	1
Yakima	1
WASHINGTON, DC	
Bolling AFB	1
Fort McNair	1
Walter Reed AMC	1
WEST VIRGINIA	
All	1
WISCONSIN	
All	1
WYOMING	
Cheyenne	1
Yellowstone	3

*Refer to table 3-1 for prescribed values of Z.

Table 3-6. Seismic Zone Tabulation, Outside U.S.[a]

Location	Zone
AFRICA:	
Algeria:	
Alger	3
Oran	3
Botswana:	
Gaberone	0
Camaroon:	
Yaounde	0
Egypt:	
Cairo	2
Port Said	2
Ethiopia:	
Addis Ababa	2
Asmara	3
Kenya:	
Nairobi	2
Liberia:	
Monrovia	1
Libya:	
Tripoli	2
Wheelus AFB	2
Malawi:	
Blantyre	3
Lilongwe	3
Zomba	3
Morocco:	
Casablanca	2
Port Lyautcy	1
Rabat	2
Tangier	3
Mozambique:	
Maputo	1
Niger:	
Niamey	0
Nigeria:	
Ibadan	0
Kaduna	0
Lagos	0
Senegal:	
Dakar	0
Somali Republic:	
Modadiscio	0
South Africa:	
Capetown	3
Durban	1
Johannesburg	2
Natal	1
Pretoria	2
Southern Rhodesia:	
Salisbury	3
Swaziland:	
Mbabane	1
Tanzania:	
Dar es Salaam	1
Zanzibar	1
Tunisia:	
Tunis	3
Uganda:	
Kampala	2
Zaire:	
Bukavu	3
Lubumbashi	2
Kinshasa	0
Zambia:	
Lusaka	2
ASIA:	
Afghanistan:	
Kabul	4
Burma:	
Mandalay	3
Rangoon	3
China:	
Canton	2
Nanking	2
Peking	4
Shanghai	2
Tihwa	4
Tsingtao	3
Cyprus:	
Nicosia	3
India:	
Bombay	3
Calcutta	2
New Delhi	3
Madras	0
Indonesia:	
Bandung	4
Jakarta	4
Medan	3
Surabaya	4
Iran:	
Isfahan	3
Shiraz	3
Tabriz	4
Tehran	3
Iraq:	
Bagdad	3
Basra	1
Israel:	
Haifa	3
Jerusalem	3
Tel Aviv	3
Japan:	
Itazuke AFB	3
Misawa AFB	3
Okinawa	4
Osaka/Kobe	4
Tokyo	4[b]
Wakkanai	3
Yokohama	4[b]
Yokota	4[b]
Korea:	
All	0
Kuwait	1
Laos:	
Vientiane	1
Lebanon:	
Beirut	3
Malaysia:	
Kuala Lumpur	1
Nepal:	
Kathmandu	4
Pakistan:	
Karachi	4
Peshawar	4
Saudi Arabia:	
Al Batin	0
Dhahran	1
Jedda	2
Jubail	1
Khamis Mushayf	1
Riyadh	0
Singapore:	
All	1
Syria:	
Aleppo	3
Damascus	3
Taiwan:	
All	4
Thailand:	
Bangkok	0
Udorn	0
Turkey:	
Ankara	2
Istanbul	4
Karamursel	3
Vietnam:	
Saigon	0
Yemen:	
Sanaa	2
ATLANTIC OCEAN AREA:	
Ascension Island	U
Azores	2
Bermuda	1
CARIBBEAN SEA:	
Bahama Islands	1
Cuba	2
Dominican Republic:	
Santo Domingo	3
Haiti:	
Port au Prince	3
Jamaica:	
Kingston	3
Leeward Islands	3
Puerto Rico	3
Trinidad	3

[a]Refer to table 3-1 for prescribed values for seismic zone nos. 0 through 4. U denotes unknown seismicity.
[b]Use local code if it is more severe than seismic zone no. 4.

Table 3-6. Seismic Zone Tabulation, Outside U.S.[*]

CENTRAL AMERICA:

Canal Zone	2
Costa Rica:	
San Jose	3
El Salvador:	
San Salvador	4
Guatemala:	4
Honduras:	
Tegucigalpa	3
Mexico:	
Ciudad Juarez	2
Guadalajara	3
Mexico City	3
Tijuana	3
Nicaragua:	
Managua	4
Panama:	
Colon	3
Panama	3

EUROPE:

Belgium:	
Antwerp	1
Brussels	2
England:	
London	2
Liverpool	1
France:	
Lyon	1
Marseille	3
Nice	3
Paris	0
Germany:	
Berlin	0
Bonn	2
Bremen	0
Dusseldorf	1
Frankfurt	2
Hamburg	0
Munich	1
Stuttgart	2
Greece:	
Athens	3
Thessaloniki	4
Iceland:	
Keflavick	3
Reykjavik	4
Thorshofn	U
Ireland:	
Belfast	0
Dublin	0
Italy:	
Aviano AFB	3
Brindisi	0
Genoa	3
Milan	2
Naples	3
Rome	2
Sicily	4
Trieste	3
Turin	2
Netherlands:	
All	0
Norway:	
Oslo	2
Portugal:	
Lisbon	4
Opporto	3
Scotland:	
Aberdeen	U
Edinburgh	1
Edzell	1
Glasgow/Renfrew	1
Londonderry	1
Prestwick	U
Shetland Islands	U
Stornoway	U
Thurso	1
Spain:	
Barcelona	2
Bilbao	2
Madrid	0
Rota	1
San Pablo	U
Seville	2
Zaragoza	U
Sweden:	
Goteborg	2
Stockholm	1
Switzerland:	
Bern	2
Geneva	1
Zurich	2

NORTH AMERICA:

Canada:	
Argentia NAS	2
Churchill, Man	0
Cold Lake, Alb	1
Edmonton, Alb.	1
E. Harmon AFB	2
Fort Williams, Ont	0
Frobisher, N.W. Ter	0
Goose Airport	0
Ottawa, Ont	2
St. John's Nfld	2
Toronto, Ont.	1
Winnipeg, Man	1
Greenland	1

SOUTH AMERICA:

Argentina:	
Buenos Aires	0
Brazil:	
All	0
Bolivia:	
La Paz	3
Santa Cruz	1
Chile:	
Santiago	4
Colombia:	
Bogota	4
Ecuador:	
Quito	4
Guayaquil	3
Paraguay:	
Asuncion	0
Peru:	
Lima	4
Piura	4
Uruguay:	
Montevideo	0
Venezuela:	
Maracaibo	2
Caracas	4

PACIFIC OCEAN AREA:

Australia:	
Canberra	
Melbourne	1
Perth	1
Sydney	1
Caroline Islands:	
Koror, Paulau Is.	2
Ponape	0
Fiji:	
Sura	3
Johnson Island	1
Mariana Islands:	
Guam	3
Kwajalein	1
Saipan	3
Tinian	3
Marshall Islands	1
Midway Island	U
New Guinea:	
Port Moresby	3
New Zealand:	
Auckland	3
Wellington	4
Philippine Islands:	
Cebu	4
Manila	4
Baguio	3
Samoa	3
Volcano Islands	U
Wake Island	0

[*]Refer to table 3-1 for prescribed values for seismic zone nos. 0 through 4. U denotes unknown seismicity.

(1) Hospitals.

(2) Fire stations, rescue stations, and garages for emergency vehicles.

(3) Power stations and other utilities required as emergency facilities.

(4) Mission-essential and primary communication or data-handling facilities.

(5) Facilities involved in operational missile control, launch, tracking or other critical defense capabilities.

(6) Facilities involved in handling, processing, or storing sensitive munitions, nuclear weaponry or processes, gas and petroleum fuels, and chemical or biological contaminants.

b. High Risk (I = 1.25). Those structures are where primary occupancy is for assembly of a large number of people, where the primary use is for people that are confined (e.g., prison), or where services are provided to a large area or large number of other buildings. Buildings in this category may suffer damage in a large earthquake but are recognized as warranting a higher level of safety than the average building. Typical examples are:

(1) Buildings whose primary occupancy is that of an auditorium, a recreation facility, dining hall, or commissary which is subject to occupancy by more than 300 persons.

(2) Confinement facilities (e.g., prisons).

(3) Central utility (power, heat, water, sewage) that are not covered by paragraph a(3) above, and that serve large areas.

(4) Buildings having high value equipment when justification provided by using agency.

c. All Others (I = 1.0). This includes all structures not covered by the above categories.

3-6. Summary of approved structural systems. The minimum values of the base shear coefficient K are set forth in table 3-3. Table 3-7 is provided as a guide to interpret table 3-3 and to summarize the approved structural systems for Seismic Zone 1, Seismic Zone 2, and Seismic Zones 3 and 4. The designations used for frame and wall specifications are described below. Note that the wall specifications include braced frames.

a. Frame Specifications. (The design requirements are covered in chapter 7.)

(1) *Concrete Frame Type A.* Ductile moment resisting space frame.

(2) *Concrete Frame Type B.* Moment resisting space frame. Qualifies as a ductile moment resisting space frame in Seismic Zone 1 only. May be used as a lateral force resisting system in Seismic Zone 2 with certain height and K limits.

(3) *Concrete Frame Type C.* Moment resisting space frame. May be used as a lateral force resisting system in Seismic Zone 1 only for buildings less than 80 feet in height.

(4) *Concrete Frame Type D.* Vertical load carrying space frame in accordance with ACI 318-77.

(5) *Steel Frame Type A.* Ductile moment resisting space frame.

(6) *Steel Frame Type B.* Moment resisting space frame. May be used as a lateral force resisting system subject to certain height and K limits.

(7) *Steel Frame Type C.* Vertical load carrying space frame in accordance with AISC Specifications. May be used as a moment resisting space frame lateral force resisting system in Seismic Zone 1 only for buildings less than 80 feet in height.

(8) *Wood frames.*

b. Wall Specifications (Includes Braced Frames). (The design requirements are covered in chapter 6.)

(1) *Shear Wall Type A.* Concrete (or steel) shear walls with vertical boundary elements.

(2) *Shear Wall Type B.* Concrete shear walls.

(3) *Braced frames.* Steel or concrete.

(4) *Masonry.* Masonry shear wall. When masonry shear walls are used as part of a dual system in Seismic Zones 2, 3, or 4, vertical boundary members are required.

(5) *Wood.* Wood stud shear walls with plywood or diagonal wood sheathing.** (*Note:* Stud wall shear walls other than those listed above limited to 2 stories with $K \geqslant 1.33$. See Stud Walls below.)

(6) *Stud walls.* Wood or metal stud walls that comply with chapter 6, paragraphs 6-5 and 6-6.

**See footnote on the bottom of table 3-3 for 1980 SEAOC modification.

Table 3-7. Approved Building Systems

Basic System[1]	K Value	Height Limit (feet)	Zone 1 Minimum Required Frame	Zone 1 Minimum Required Wall	Zone 2 Minimum Required Frame	Zone 2 Minimum Required Wall	Zones 3 and 4 Minimum Required Frame	Zones 3 and 4 Minimum Required Wall
Frames (100% of Force in Frame), Categories 1 and 2	0.67	None	Concrete B or Steel A		Concrete A or Steel A		Concrete A or Steel A	
	1.00	160	Steel B		Steel B		Not Applicable	
		80	Concrete C or Steel C		Concrete B or Steel B		Steel B	
Dual Systems (Frame 25%, Wall 100%), Category 3	0.80	None	Concrete B or Steel A	Shear Wall A or Braced Frame	Concrete A or Steel A	Shear Wall A or Braced Frame	Concrete A or Steel A	Shear Wall A or Braced Frame
		160	Steel B	Shear Wall A or Braced Frame	Concrete B or Steel B	Shear Wall A or Braced Frame	Not Applicable	Not Applicable
		80	Concrete B or Steel B	Shear Wall B or Masonry	Concrete B or Steel B	Masonry[2]	Concrete A or Steel A	Masonry[2]
Shear Walls or Braced Frames (100% of Force in Wall), Categories, 4, 5, and 6	1.00	None	Concrete D[3] or Steel C	Shear Wall B or Braced Frame	Not permitted over 160 feet			
		160	Not Applicable	Not Applicable	Concrete D[3] or Steel C	Shear Wall A or Braced Frame	Concrete D[3] or Steel C	Shear Wall A or Braced Frame
		80	Concrete D[3] or Steel C	Masonry	Concrete D[3] or Steel C	Shear Wall B or Masonry	Concrete D[3] or Steel C	Shear Wall B or Masonry
		40 3 Stories		Wood **		Wood**		Wood**
	1.33	None		Shear Wall B or Braced Frame	Not permitted over 160 feet			
		160		Not Applicable		Shear Wall A or Braced Frame		Shear Wall A or Braced Frame
		80		Masonry		Shear Wall B or Masonry		Shear Wall B or Masonry
		2 Stories		Stud Walls[4]		Stud Walls[4]		Stud Walls[4]

[1]Categories as defined in Table 3-3.
[2]Vertical boundary elements in accordance with Chapter 6, paragraph 6-8.
[3]Frames required for gravity loads only. See requirement c of Table 3-3, category 4.
[4]Wood frame or stud wall construction not in accordance with requirement b of Table 3-3, category 5.
**See footnote on the bottom of Table 3-3 for 1980 SEAOC modifications.

CHAPTER 4
DESIGN PROCEDURE

4-1. Purpose and scope. This chapter describes a general procedure for the design of buildings to resist the earthquake lateral forces specified in chapter 3, Design Criteria. Procedures for designing and detailing of structural elements of buildings are more fully discussed in chapters 5 through 8. Detailed examples for specific types of structures are included in the appendices of this manual.

4-2. Preliminary design. The preliminary seismic design of the structure requires site investigations, conceptual planning with the architect and the mechanical and electrical engineer, selection of a workable structural system, and selection of trial member sizes.

a. Site Investigation. Before proceeding with the design of a building, the engineer must know the seismic zone, the foundation conditions and hazards, and the tsunami generation characteristics (refer to chap 2, para 2-7). In some cases geotechnical data may be required to determine T_s (refer to para 4-3*f*).

b. Conceptual Planning. Collaboration of the architect and structural, mechanical, and electrical engineers is required to establish a concept for the overall building system, to select the materials of construction, and to reconcile the conflicting requirements of architectural, structural, mechanical, and electrical sytems (refer to chap 2, para 2-8).

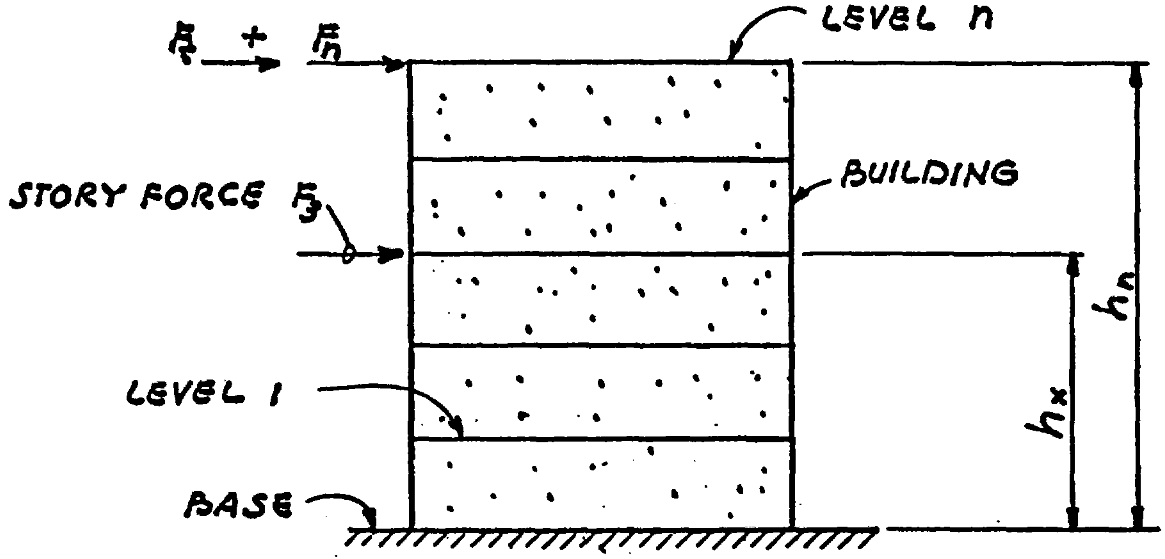
c. Selection of Structural System. Before selecting the structural system, a familiarity with the techniques and application of seismic design is essential (refer to chap 2, para 2-9). Also the possibility of future expansion must be considered (chap 2, para 2-11). The limitations on structural systems (chap 3, para 3-3 and 3-6) and the special requirements for ductility, tall buildings, concrete frames, braced frames, shear walls, concrete and masonry, diaphragms, foundations, and exterior elements (chap 3, paragraph 3-3(J)) must be reviewed.

d. Selection of Trial Structural Member Size. Some of the structural members of a building are governed by the gravity load design and are not affected by the seismic loads. For these members the sizes will have been determined by the usual requirements for dead and live loads. For the sizes of members that form the seismic lateral force-resisting system, a trial and error process is required because of the magnitude of the design forces depends on the period of the building while the period depends on the weight and stiffness of the building. First, trial design lateral forces are obtained from approximate calculation of period and weight. Next, trial member sizes are selected using approximate calculations and judgment. Finally, a preliminary analysis is made, and the trial sizes are confirmed or revised. If there are substantial revisions to the initial trial sizes, the response characteristics of the structure will change and a reanalysis may be required.

4-3. Minimum earthquake forces. Every building will be designed for lateral seismic forces, acting nonconcurrently in the direction of each main axis of the structure (also, see para 4-4*c*). As a minimum, the total forces (V = ZIKCSW) specified in chap 3, para 3-3(D), will be applied to the structure as a whole and will be distributed to the various levels of the structure as prescribed in chapter 3, paragraph 3-3(E). The coefficients Z, I, K, C, and S depend on the seismic zone of the site, occupancy importance, type of lateral resisting system (e.g., shear wall or space frame), the period of the structure, and the site characteristics, respectively. W is the effective weight of the structure. These, as well as other symbols, are defined in chapter 3, paragraph 3-3(C), and methods for determining their values are discussed below. Some basic terminology is defined in chapter 3, paragraph 3-3(B). A graphic representation of seismic forces is shown in figure 4-1. The product of ZIKCS can result in an upper limit of 0.28 for buildings in zones of the highest seismicity. The lower limit for ZIKCS in any of the four seismic zones is 0.015.

a. Z-Factor. The factor Z, which represents the seismicity of the site, is equal to or less than 1.0. It is obtained from chapter 3, table 3-1, and is dependent on the seismic zone maps of chapter 3, paragraph 3-4. For California and Nevada use the map in figure 3-4; the other Contiguous States, Alaska, and Hawaii use the maps in figures 3-1, 3-2, and 3-3, respectively. Seismic zones for specific areas within the United States are tabulated in table 3-5. For localities outside the United States refer to the tabulation in figure 3-6. The boundary lines are approximate. If there is some uncertainty about the location or the seismicity of the site, the larger number will be used.

b. I-Factor. The value of the factor I is determined from the occupancy classifications of chapter



BASE SHEAR ←——, V (TOTAL FORCE)

BASE O.T.M., (OVERTURNING MOMENT)

$F_t = 0.07\,TV$ .... (3-6)	$V = ZIKCSW$ (3-1)
$F_n = \dfrac{(V-F_t)\,w_n h_n}{\sum_{i=1}^{n} w_i h_i}$ ---- (3-7)	$= F_t + \sum_{i=1}^{n} F_i$ (3-5)
$F_x = \dfrac{(V-F_t)\,w_x h_x}{\sum_{i=1}^{n} w_i h_i}$ ---- (3-7)	$OTM = (F_t + F_n)\,h_n + \sum_{i=1}^{n} F_i h_i$ NOTE: IN SOME CASES WIND MAY GOVERN

SUBSCRIPT DESIGNATIONS

n = NUMBER OF STORIES. IN THIS EXAMPLE $n = 5$

x = THE STORY LEVEL UNDER CONSIDERATION AS IN THE FORCE F_3 AT LEVEL $x = 3$

i = STORY LEVELS USED IN SUMMATIONS RANGING FROM $i = 1$ AT THE FIRST LEVEL ABOVE THE BASE TO $i = n$ AT THE UPPERMOST LEVEL

Figure 4-1 Seismic forces

3, table 3-2. The values range from 1.0 to 1.5. Examples of various occupancy classifications are given in chapter 3, paragraph 3-5. When there is some doubt regarding the proper value of the I-factor, the decision will be made by the Design Agent.

c. *K-Factor.* The factor K represents the type of structural system and the nature of the structure itself. The value of K, which is obtained from chapter 3, table 3-3, varies from 0.67 to 1.33 for buildings and from 2.0 to 2.5 for structures other than buildings. Buildings that are considered to possess considerable inelastic deformation ability and/or have inherent redundancy are assigned the lower K values. Buildings that tend to be more brittle and lack redundancy are assigned the higher K-values. Damping, to a certain extent, is also considered in the K-value. Whereas buildings generally have a multiplicity of nonstructural and noncomputed resisting elements that effectively increase the resistance of the structure, structures other than buildings generally do not have such elements or have low damping characteristics and are assigned larger K-values. A summary of approved structural systems for each of the seismic zones is provided in table 3-7 of chapter 3. Although the selection of the K-factor is generally a simple process, for some buildings it may be complicated by unusual combinations of materials, height limitations, ductility requirements, and other special requirements. In the following paragraphs several of the parameters that influence the K-factor are discussed as a guide to selecting the proper value.

(1) *Seismic zone.* The requirements for the K-values vary slightly for the different seismic zones. In Zone 1, there are fewer restrictions on buildings over 160 feet in height. In Zones 1 and 2 there are fewer requirements on ductility for frames.

(2) *Height of building.* Some approved structural systems are restricted by height limitations. Buildings over 160 feet in height must be ductile moment-resisting space frames (K = 0.67) or dual systems (K = 0.80); however, some exceptions are allowed for Zone 1. Some space frames that do not satisfy special ductility requirements are limited to 80 feet; reinforced masonry walls are limited to 80 feet in height; and wood buildings are limited to three stories or 40 feet in height.

(3) *Combinations of K-values.* If K = 1.33 is used in one direction of a building, it must be used in both directions. For other values of K, it need not be the same in both directions. Generally the K-value is constant throughout the height of the building. When a change of structural system does occur (e.g., steel frame on concrete shear walls, wood box system on a concrete box system), the K-value at the lower level cannot be less than the K-value of the system above, and special consideration must be given to the transition from one system to the other to assure sufficient load transfer capacity and inelastic deformation capability.

(4) *Vertical load-carrying system.* If the building does not have a complete vertical load-carrying space frame, it is considered to be a box system and has K = 1.33. In other words, if shear walls are used to support the vertical floor loads, K = 1.33. In order to use a value of K less than 1.33, the building must have a vertical load-carrying space frame that is designed to carry essentially all vertical loads. However, some exceptions are acceptable such as a minor load-bearing wall that does not significantly influence the lateral force characteristics of the building. Also, basement walls below the level considered as the base of the building may be bearing for loads originating at such level. The test for qualifying as a vertical load-carrying space frame is to determine whether or not the building can support the vertical loads if the shear walls are seriously damaged during an earthquake.

(5) *Lateral force-resisting system.* The lateral force-resisting system for a building is either (a) a box system (table 3-3, Categories 5 and 6, shear walls or braced frames without a complete vertical load-carrying space frame), (b) a shear wall (or braced frame) system with a nonseismic-resisting, vertical load-carrying space frame (table 3-3, Category 4), (c) a dual system consisting of both shear walls (or braced frames) and a lateral force-resisting frame (table 3-3, Category 3), or (d) a space frame system—ductile moment-resisting or moment-resisting types (table 3-3, Categories 1 and 2). These lateral force-resisting systems are reclassified in table 3-7 to account for the various requirements in the different seismic zones.

(6) *Buildings not classified above.* Any building designed within the scope of this manual must qualify under one of the classifications defined in chapter 3, table 3-3, or table 3-7, or discussed above. If there is doubt as to which of two classifications govern, the one with the larger value of K should be used. If the building does not appear to be covered by any of the classifications, the structural system must be modified to conform to one of the classifications or justification must be made that the structural system will satisfy the intent of the seismic design provisions.

d. *T, Building Period.* The period of vibration, T, is the time required for one complete cycle of oscillation of an elastic structure in a particular mode of vibration. The building period referred to in the seismic provisions of this manual is the fundamental period of vibration for each of the two translational directions of the building (e.g., transverse and longitudinal directions). In the fundamental mode the building acts as a cantilever essentially fixed at the base, swaying first to one side and then to the other side. The calculation of the period, in accordance with formula 3-3, requires a knowledge of the lateral stiffness characteristics of building (i.e., force versus displacement relationship). The fundamental period of vibration, T, in each direction of the proposed structure, is required in order to determine the C-factor, the S-factor, and in some cases to determine the force distribution, F_t, at the top of the structure. Because the above factors must be known during the initial design stage when the sizes and details of all the structural elements may not have been established (thus the stiffness characteristics are not know), an estimated initial value of T must be used. The estimated value need only be accurate enough to establish reasonable values for C, S, and F_t. The product of CS will be underestimated if the assumed building period is too long, therefore, the estimated period should be on the short side in order to be conservative. At the final design stage, the period must be checked so that C and S values used in the design are either conservative or consistent with the final period. The sensitivity of these factors is discussed in more detail in paragraphs 4-3*e*, *f*, and *g* and 4-4*a*.

(1) *Period for low-rise buildings.* For most low-rise buildings (e.g., up to 5 stories with periods shorter than 0.5 second) the calculation of T is not necessary because C and CS are at their maximum values and F_t is equal to zero. Refer to paragraph 4-3*g* for additional discussion.

(2) *Initial period estimation.* As an initial step to estimate the building period in the fundamental mode, the use of Formulas 3-3A and 3-3B, as specified in chapter 3, paragraph 3-3(D), is acceptable. These empirical formulas rely only on basic building dimensions and the number of stories so that they are easy to apply at the initial stage of the design. The resulting period is generally shorter than the actual period; thus it can be safely used for the final design. However, if feasible, a more accurate estimate of the period should be made after the member sizes of the lateral-resisting system have been determined.

(3) *Alternate method for initial period estimation.* For some structures, member sizes are controlled by limits on lateral drift (e.g., chap 3, para 3-3(H)1) rather than by stress limitations. This condition generally applies to structural steel mome resisting space frames systems with nonpart pating walls and partitions. If the drift limitations are used as a basis for determining a predesign initial period estimation, precautions must be observed in order not to underestimate the total lateral force by estimating a period that is longer than the actual period. After member sizes have been determined, the period must be recalculated as described in paragraph (4). The limiting values of paragraph (5) will be applicable (refer to Design Example A-3).

(4) *Period calculation.* When formula 3-3 is employed (see fig 4-2), the most difficult part involves the determination of the story displacements (δ_i). The story weights (w_i) are relatively simple to estimate, and almost any set of story forces (f_i) can be used (e.g., the inverted triangular distribution such as obtained from formula 3-7 usually gives good results), but the corresponding lateral story displacements must be calculated. The basic objective must be a realistic approach to calculating the actual period—rather than the manipulation of the structure model so as to obtain a "calculated" but nonvalid long period and low base shear. For simple structures, the lateral displacements required for Formula 3-3 can be obtained by hand calculation methods. For complex structures, the calculatic for lateral displacements become lengthy so that aid of a computer program is normally used. Som programs that calculate member forces and frame deflections include a calculation of periods and mode shapes. Calculations must take into account all elements which stiffen the structure even if they are not part of the seismic-resisting system. (*Note:* The assumption for the stiffer structure is used to calculate the period for determination of lateral force coefficients, but it is unconservative to use this assumed stiffness to satisfy drift requirements as discussed in para 4-5c.)

(5) *Maximum value for period.* Using an unrealistically long period for calculating the coefficients C and S can result in an unconservative design. Because of the many parameters involved, it is difficult to establish a hard and fast rule for what the maximum value of the period T should be. The SEAOC Commentary advises a thorough examination if the calculated T exceeds $0.5N^{2/3}$, where N is the number of stories above the base to level n. This formula results in periods ranging from 0.8 second for a two-story building to 3.0 seconds for a 15-story building. Even these periods are felt by some engineers to be too long. The Applied Technology Cr cil (ATC), in publication ATC 3-06, "Tentative visions for the Development of Seismic Regulations

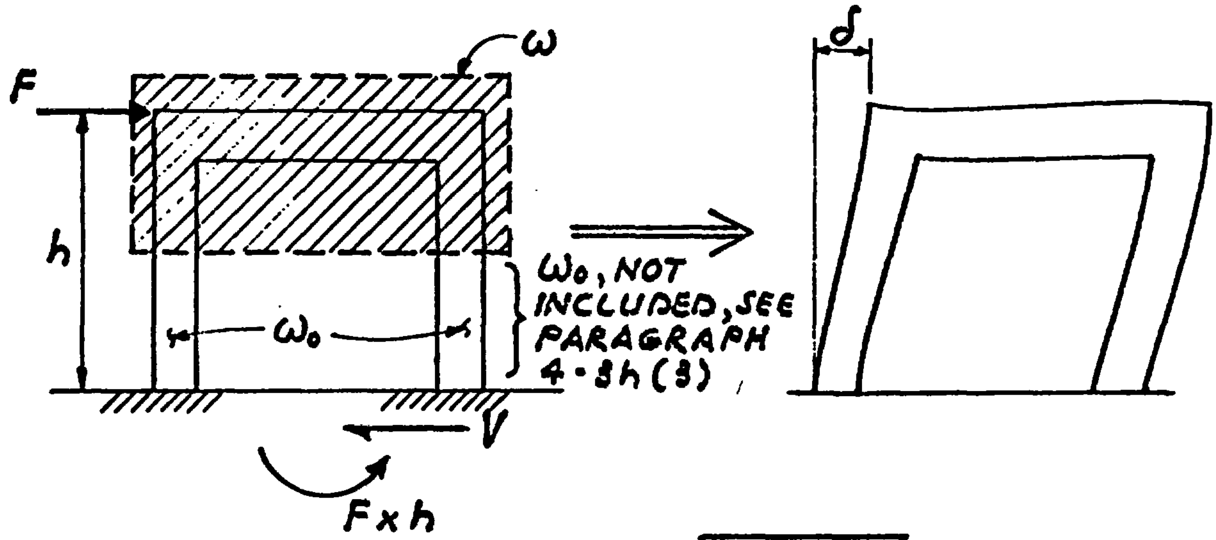


FIGURE 4-2a $$T = 2\pi\sqrt{\frac{\omega \times \delta^2}{gF\delta}} \qquad (3\text{-}3)$$

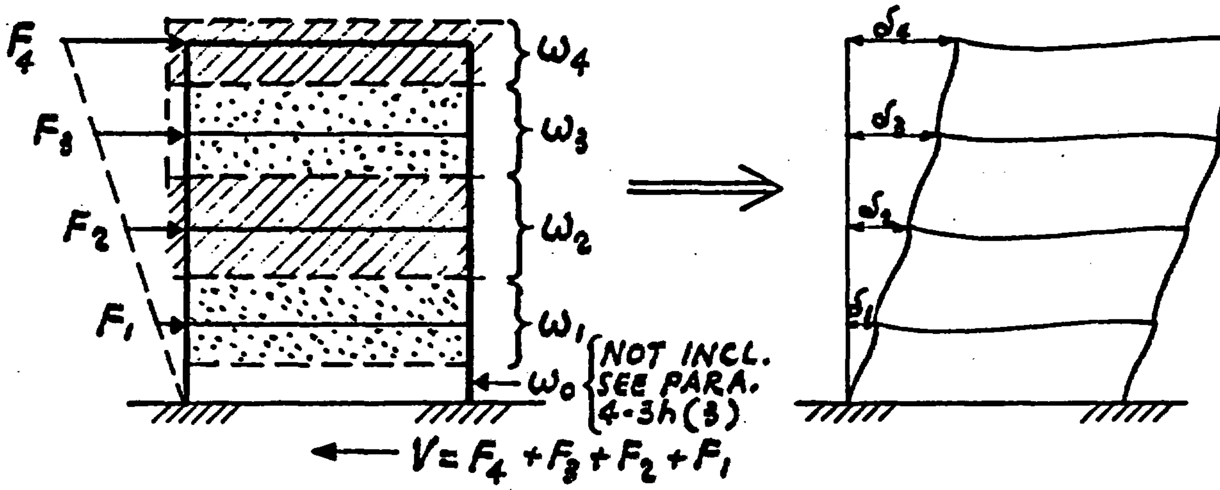


FIGURE 4-2b $$T = 2\pi\sqrt{\frac{1}{g}\left[\frac{\omega_4\delta_4^2 + \omega_3\delta_3^2 + \omega_2\delta_2^2 + \omega_1\delta_1^2}{F_4\delta_4 + F_3\delta_3 + F_2\delta_2 + F_1\delta_1}\right]} \qquad (3\text{-}3)$$

Figure 4-2 Period calculation

for Buildings,"* recommends that the minimum design lateral force be based on a maximum value of T equal to 1.4 $C_R h_n^{3/4}$, where h_n is the height of the building in feet and $C_R = 0.025$ for concrete frames and $C_R = 0.035$ for steel frames.* For steel frame buildings, the formula results in periods ranging from 0.5 second for a two-story (24 feet) building to 2.4 seconds for a 15-story (180 feet) building. In this manual the ATC formula is suggested as a limiting value for the period T for use in calculating the C and S coefficients, in lieu of more current data. However, the designer must not use the above formulas for estimating the period used in design. The formulas are only to be used to check against the value of T calculated from the actual building properties.

e. C-Factor. The factor C is dependent on the period T of the structure as shown in formula 3-2, chapter 3. The maximum value of C is 0.12, which occurs for all values of T less than 0.31 second. At the other extreme range of the scale, where T is 5.0 seconds (say a 50-story building), the value of C is 0.03 or about one-fourth of the maximum value. Table 4-1 below gives some values for C as a function of T. This table may be used in lieu of formula 3-2. The factor S is also dependent on the period T. Refer to paragraph 4-3*g* for combined CS factors.

f. S-Factor and T_s. The factor S is dependent on the ratio of building period (T) to characteristic site period (T_s) as shown in formulas 3-4 and 3-4A, chapter 3. The value of S may vary from 1.0 to 1.5. The maximum value occurs when $T = T_s$. To use less than the maximum S, values for both T and T_s must be substantiated. For guidelines for determining T, refer to paragraph 4-3*d* above. In order to determine a value for T_s, a geotechnical investigation may be necessary (for guidelines for determining T_s, refer to "SEAOC Standard No. 1, Determin of the Characteristic Site Period, T_s," App B of the SEAOC Recommendations). However, for most low-rise buildings (e.g., T < 0.3 second), where the difference between the minimum and maximum effective S value is only 5 percent, the maximum value is used and T_s need not be determined (refer to para 4-3*g*). For taller buildings, where T_s can affect the base shear coefficient by as much as 50 percent, it may be worthwhile to have a geotechnical investigation made. On some sites the values of T_s may be obvious without a detailed investigation. For example, if the building is to be located on a firm site, T_s will be 0.5 second. A firm site is defined as a site where bedrock is within 10 feet or where there is very dense granular soils. At the other end of the scale, where there may be over 500 feet of dense sand or over 300 feet of consolidated clay, T_s may be about 2.5 seconds. When a geotechnical investigation is made, T_s might not always be presented as a simple value, but might be represented by a reasonable range of values. When this occurs, the building period must be compared with the range of T_s values to obtain the highest value for S.

(1) *Example for T_s given as a range of values.* If T_s is given to be in the range of 1.0 second to 1.5 seconds, then:

(a) For a building with a period shorter 1.0 second, use a T_s value of 1.0 second.

(b) For a building with a period longer than 1.5 seconds, use a T_s value of 1.5 seconds.

(c) For a building with a period within the range of 1.0 to 1.5 seconds, T/T_s will be taken to equal 1.0 and S will equal 1.5.

(2) *Table for S-factor.* Table 4-2 below gives some values of S as a function of T/T_s. This table can be used in lieu of formulas 3-4 and 3-4A. Refer to paragraph 4-3*g*, below, for CS factors combined.

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Table 4-1. $C = 1/15\sqrt{T}$ (3-2)

T	<0.31	0.40	0.50	0.75	1.00	1.25	1.50	2.00	3.00	5.00
C	0.120	0.105	0.094	0.077	0.067	0.060	0.054	0.047	0.038	0.030

*In the ATC publication, the 1.4 coefficient is applicable to the modal analysis procedure (ATC Sec. 5.8) and a coefficient of 1.2 is recommended for the equivalent lateral force procedure (ATC Sec. 4.2.2).

Table 4-2. S as a Function of T/T_s

T/T_s	0.12	0.20	0.30	0.40	0.60	0.80	1.00	1.20	1.60	2.00	>2.29
S	1.11	1.18	1.26	1.32	1.42	1.48	1.50	1.49	1.40	1.20	1[illegible]

(3) *T and T_s limitations* for the calculation of S.

(a) If the period of the building is shorter than 0.3 second, use T = 0.3 second.

(b) T_s will range from 0.5 second to 2.5 seconds.

(c) If T is longer than 2.5 seconds and T_s is unknown, use T_s = 2.5 seconds.

g. Combined CS Factors. The product of C and S factors describes the general relationship of base shear coefficients to building period of vibration (Z, I, and K are independent of T). CS ranges from a maximum of 0.140 for short period buildings to a value of 0.027 for a building with a period of 6 seconds (such as for a 60-story building). Table 4-3 gives some values of CS as a function of building period (T) and site characteristic period (T_s). Figure 4-3 illustrates the relationship of CS to T graphically, showing the maximum and minimum CS values. Note that for some building periods, CS is not very sensitive to a variation in T_s.

h. Weight. W, the total dead load and applicable portions of other loads, represents the total mass of the building. It includes the weight of the structural slabs, beams, columns, and walls as well as nonstructural components such as partitions, ceilings, floor topping, roofing, fireproofing material, and fixed electrical and mechanical equipment. When partition locations are subject to change, a uniform distributed dead load of 20 pounds per square foot of floor space is used. Miscellaneous items such as ducts, typical piping, and conduits can be covered by an additional 1 or 2 pounds per square foot. In storage areas, 25 percent of the design live load shall be included in the seismic weight W. In areas of heavy snow loads, some or all of the design snow load must be included (refer to chap 3, para 3-3(D)5c). At the initial stage of design, the estimated weights of the structural members will be used. After the final sizes of structural members are selected, the actual weights must be compared with the estimated weights. In addition to determining the overall weight W, the designer must determine tributary weights at each floor for both vertical and horizontal distribution. Therefore, the calculations for W must be done in an orderly manner so that tributary weights as well as the overall weights can be accounted for.

(1) *Vertical distribution.* For vertical distribution, the weight "w_x" that contributes to story level "x" is calculated separately for each floor (refer to chap 3, para 3-3(E)). This generally includes the weight of the complete floor system, plus one-half the weight of the story walls and columns above the floor level and one-half of the weight of the story walls and columns below the floor level. If partitions are laterally supported top and bottom, their weight is divided between the two floor levels; however, if the partitions are free standing, the total weight is included with the supporting floor level.

(2) *Horizontal distribution.* The horizontal distribution of weight at each floor level is required in order to calculate the center of mass (chap 3, para 3-3(E)5) and the diaphragm forces (chap 3, para 3-3(J)2d). The weight of the diaphragm and the elements tributary thereto (designated w_{px} in formula 3-9) include the floor system, tributary weights of walls and partitions, and other elements attached to the diaphragm. The weights of the shear walls (and items attached thereto) that act in the same direction under consideration for the diaphragm, need not be included in the weight of the diaphragm unless there is vertical discontinuity such that redistribution of the shear wall weight to other shear walls is required. The horizontal distribution generally consists of a combination of uniform and concentrated weights along the length of the floor plus concentrated weights tributary to the shear walls at the shear walls (see fig 4-4).

(3) *Summation.* The sum of the horizontal distribution weight (in each direction of motion) will be equal to the story weight, and the sum of the story weights equal the total weight W of the building, except that the bottom half of the first story generally distributes itself directly to the base and is not necessarily included in the weight W (fig 4-2).

4-4. Distribution of forces. The total lateral force is distributed throughout the building in a manner that simulates the behavior of the building during an earthquake.

a. Story Forces. The distribution of the lateral force vertically along the height of the building is determined by formula 3-7 (fig 4-1) except for those buildings that are considered irregular. A sample format for determining story forces is shown in table 4-4. The procedure given is based on the assumption of a uniform building and is aimed at a reasonable evaluation of the relative maximum story shear (e.g., column (9) in table 4-4) envelope that will occur.

(1) *Regular buildings with T < 0.7 second.* When the period of the building is less than 0.7 second, F_t will be equal to zero. Then formula 3-7, the vertical distribution equation, will reduce to the following:

$$F_x = \left(\frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \right) V \qquad (4\text{-}1)$$

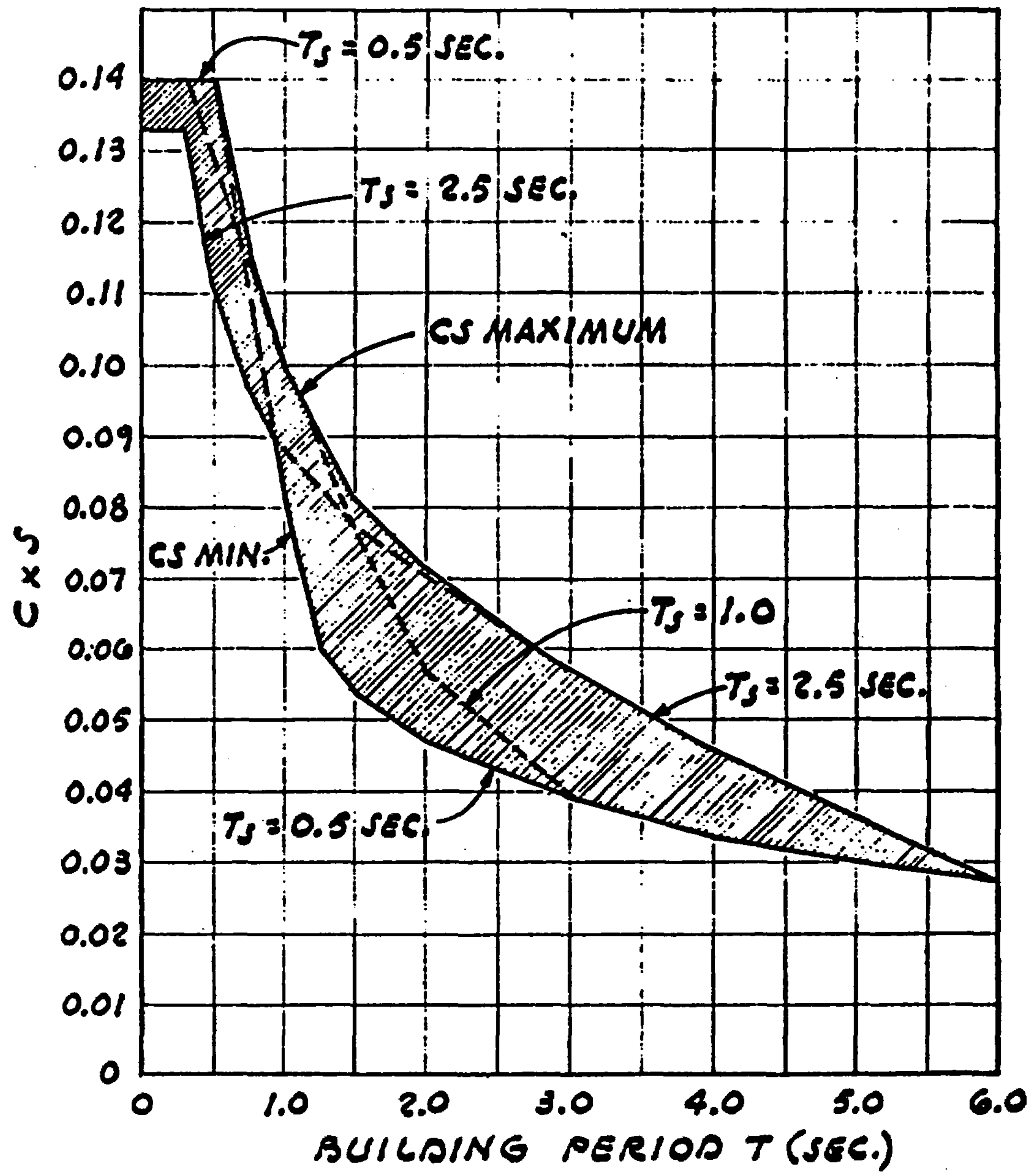


Figure 4-3. CS vs T

Table 4-3. CS as a Function of T and T_s

T_s \ T	<0.3	0.5	0.75	1.00	1.25	1.50	2.00	3.00	4.00	5.00	6.00
0.5	.140	.140	.110	.080	.060	.054	.047	.039	.033	.030	.027
0.75	.140	.136	.116	.098	.082	.065	.047	.039	.033	.030	.027
1.00	.140	.130	.113	.100	.089	.077	.057	.039	.033	.030	.027
1.25	.140	.124	.109	.099	.090	.080	.065	.039	.033	.030	.027
1.50	.140	.120	.106	.096	.089	.081	.069	.046	.033	.030	.027
1.75	.140	.117	.103	.094	.088	.080	.070	.052	.033	.030	.027
2.00	.137	.115	.100	.092	.086	.079	.071	.055	.040	.030	.027
2.50	.133	.111	.097	.088	.083	.077	.070	.057	.046	.036	.027
Unknown	.140	.140	.116	.100	.090	.081	.071	.057	.046	.036	.027

FOOTNOTES TO TABLE 4-3

(1) *If T is shorter than 0.3 seconds.* This category covers most shear wall buildings up to four stories and frame structures up to two or three stories. When T is less than 0.3, the product of CS ranges from 0.133 to 0.140. Unless T_s is known to be longer than 1.75 seconds, use CS = 0.14.

(a) At this period range, C equals 0.12.

(b) The effective value of S ranges from 1.11 to 1.17. There is only a 5 percent difference between maximum and minimum. The minimum value of T/T_s equals 0.3/2.5 equals 0.12; thus, from table 4-2, the minimum S equals 1.11. The maximum value of CS is 0.14 and C is equal to 0.12; thus, the maximum value of S equals 0.14/0.12 equals 1.17.

(c) Some low rise moment resistant steel space frames may have calculated periods greater than 0.3 second. If the longer periods are substantiated, a smaller value for CS may be justified. Refer to paragraphs 4-3*d*(4), (5) for period calculations and limiting values.

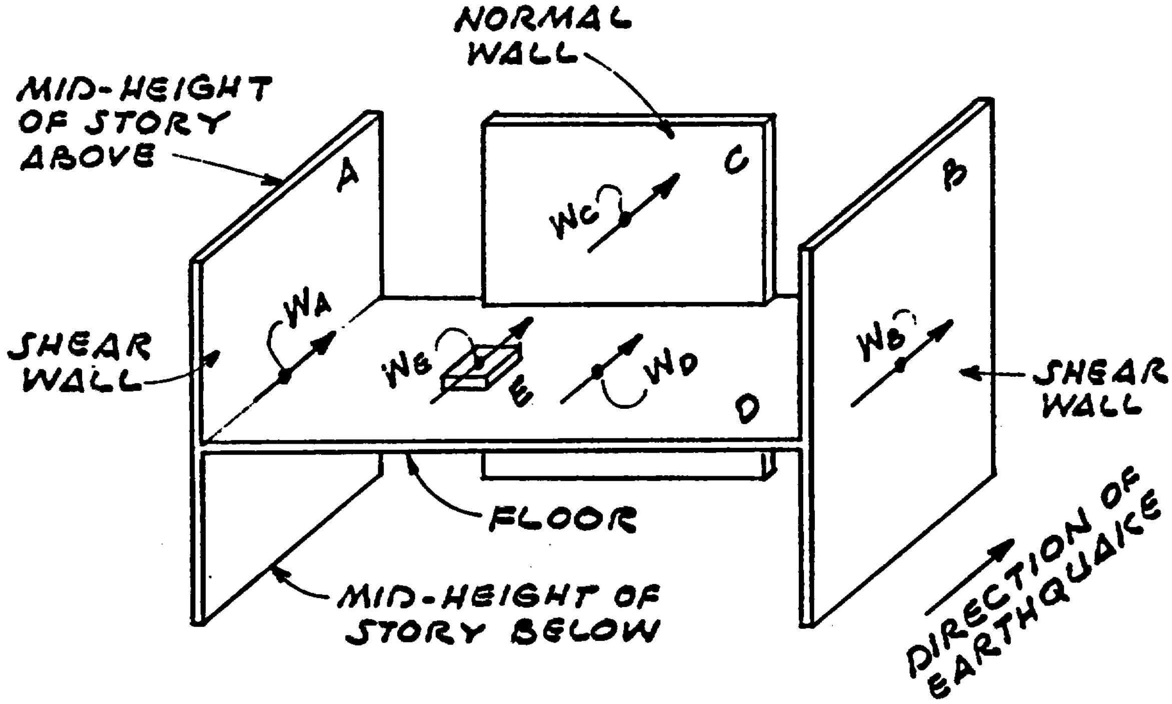
(2) *If T is about 0.5 second.* This category generally covers shear wall buildings in the order of seven stories with a 50-foot base dimension or 10 stories with a 100-foot base dimension and frame structures up to five stories. CS ranges from 0.111 to 0.140. If T_s is unknown or if the building is located on a relatively firm site, use CS equal to 0.14. It if appears that T_s may be somewhat greater than 1.0 second, it may be worthwhile to substantiate a value for T_s in order to use a value of CS less than 0.14.

(3) *If T is between 0.5 second and 1.0 second.* In this period range the values of CS are quite sensitive to period variations, ranging from 0.14 to 0.08 (fig 4-4). The value of S will range from 1.2 to 1.5, depending on the various combinations of T and T_s. The value of C will range from 0.094 to 0.067 (table 4-1).

(4) *If T is 1.0 to 1.5 seconds.* Unless it can be substantiated that the building is located on a firm site (e.g., T_s less than 0.6T), the CS value will be within about 10 percent of the maximum values shown in table 4-3.

(5) *If T is 2.0 to 4.0 seconds.* In this building period range, the difference between a firm site and a soft site can affect CS by a factor of 1.5; therefore, the costs of substantiating the value of T_s may be justified.

(6) *If T is greater than 5 seconds.* When the building period is longer than 5.7 seconds, S equals 1.0 and T_s has no effect on the value of CS.



STORY WEIGHT FOR CALCULATION OF LATERAL FORCES:

$$W_x = \text{WALLS} + \text{FLOOR} + \text{EQUIPMENT}$$

$$= W_A + W_B + W_C + W_D + W_E$$

WEIGHT FOR DESIGN OF DIAPHRAGM

$$W_{px} = \text{NORMAL WALLS} + \text{FLOOR} + \text{EQUIPMENT}$$

$$= W_C + W_D + W_E$$

NOTE:

FLOOR WEIGHT W_D, INCLUDES FLOOR STRUCTURE, SUSPENDED CEILING, MECHANICAL EQUIPMENT (UNLESS TAKEN SEPARATELY AS W_E), AND (IF APPLICABLE) 20 PSF FOR PARTITIONS.

Figure 4-4 Tributary weights at a story

Table 4-4 Force distribution

BUILDING : SEVEN STORY BLDG. $\quad$ W = 10,540 Kips

DIRECTION : LONGITUDINAL (N-S) $\quad$ T = 0.8 Sec.

Z = 1.0 $\quad$ I = 1.0 $\quad$ K = 0.67 $\quad$ C = 0.075 $\quad$ S = 1.5 $\quad$ ZIKCS = 0.075

$V = ZIKCSW = 791\ KIPS \quad F_t = 0.07TV = 0.056V \quad F_x = (V - F_t)\frac{wh}{\Sigma wh} = 0.944V\frac{wh}{\Sigma wh}$

LEVEL (1)	h FT. (2)	Δh FT. (3)	w KIPS (4)	Σw (5)	(2)×(4) wh (6)	$\frac{wh}{\Sigma wh}$ (7)	F KIPS (8)	Σ(8) V KIPS (9)	(3)×(9) ΔOTM KIP-FT (10)	Σ(10) OTM KIP-FT (11)	(9)÷(5) $\frac{F_t+\Sigma F_i}{\Sigma w_i}$ (12)*
						F_t =	44				
ROOF	65.7		1410		92637	0.228	170				
		8.7		1410				214	1862		0.152
7	57.0		1460		83220	0.205	153			1862	
		8.7		2870				367	3193		0.128
6	48.3		1460		70518	0.174	130			5055	
		8.7		4330				497	4324		0.115
5	39.6		1460		57816	0.142	106			9379	
		8.7		5790				603	5246		0.104
4	30.9		1460		45114	0.111	83			14625	
		8.7		7250				686	5968		0.095
3	22.2		1460		32412	0.080	60			20593	
		8.7		8710				746	6490		0.086
2	13.5		1830		24705	0.061	46			27083	
		13.5		10540				792	10692		0.075
GRD.	0									** 37775	* FOR USE IN FORMULA 3-9
Σ			10,540		406,422	1.001	792		37,775		

** FOR FOUNDATION OVERTURNING MOMENTS, THIS VALUE MAY BE REDUCED BY 2891[K'] (44 × 65.7) WHEN F_t IS NEGLECTED. SEE PARAGRAPH 4-4a(3).

The story force F_x is distributed horizontally at level x in proportion to its mass distribution at that level (refer to para 4-3*h*(2) and fig 4-4).

(2) *Regular building with T>0.7 second.* When the period of the building is greater than 0.7 second, a lateral force F_t, as determined by formula 3-6, is applied to the top level of the structure, usually the roof. F_t will vary from 5 percent (T = 0.7 second) to 25 percent (T≥3.6 seconds) of the lateral force V. The remaining portion of the force (V - F_t) is distributed throughout the height of the structure in accordance with formula 3-7. The total applied force at the top level of the structure will be $F_t + F_n$, where F_n is the value of F obtained from formula 3-7 for the top level "n" (see fig 4-1).

(3) *Additional comments on F_t.* The rationale for F_t is based on the following assumption: For buildings with periods greater than 0.7 second (e.g., tall and/or flexible structures), the fundamental mode shape may depart from the straight-line assumption (formula 4-1) and the effects of higher modes of vibration may become more significant. To account for this, a greater portion of the lateral force is assigned to the top of the structure by use of F_t from formula 3-6. This additional force is intended to increase the shear force and the equivalent story acceleration at the upper stories; however, in some cases the strict application of F_t may result in excessive forces for roof diaphragms and excessive overturning moments at foundations. To lessen these effects for diaphragms, chapter 3, paragraph 3-3(J)2d, places a limit of $0.30ZIw_{px}$ on the required diaphragm force; and for overturning moments at foundations, the SEAOC Commentary suggests that F_t may be neglected. A better approximation of the force distribution may be made by using the principles of dynamics which include the significant modes of vibration (see para (4) below).

(4) *Irregular and setback buildings.* For irregular structures or framing systems (chap 3, para 3-3(E)3) or for setback buildings (chap 3, para 3-3(E)2), the lateral force cannot be distributed in accordance with the arbitrary rules for uniform buildings that are contained in formulas 3-6 and 3-7, but must be distributed by a rational procedure that takes into account the stiffness properties of the lateral force resisting system, the mass distribution, and the principles of dynamics. Refer to SEAOC Recommendations, appendix C, for proposed provisions on setback buildings. Conditions of irregularity that require special design procedures include the following:

(a) Buildings with irregular configuration in plan or in the vertical dimension (e.g., L-, U-, and T-plan and setback buildings).

(b) Buildings with abrupt changes in lateral resistance within any level or between adjacent levels (e.g., discontinuity of shear walls or columns).

(c) Buildings with abrupt changes in lateral stiffness within any level or between adjacent levels (e.g., large change in size of shear walls or column piers).

(d) Unusual or novel structural features.

b. Overturning. The overturning effects are determined by applying the story forces obtained from formulas 3-6 and 3-7 as illustrated in table 4-4 and figure 4-1. The structure must resist these forces in accordance with chapter 3, paragraph 3-3(F). In moment-resistant frame structures, the overturning is resisted by a combination of coupled axial column forces and bending moments in the column. In shear wall buildings, the overturning moments are resisted by bending in the shear walls. When shear walls are linked together by beams, axial forces are transmitted to the shear walls. The distribution between the resisting axial overturning forces and bending moments are dependent on the relative stiffnesses of horizontal and vertical structural elements. Accurate determination of the resisting forces can be complex; therefore, approximate methods are generally used. One method may be used for calculating the axial forces and another method may be used for calculating bending moments and shears to assure that the structural elements are not underdesigned. The forces for the columns and shear walls must be transmitted to the foundations. In zones of high seismicity, the application of the design forces create an apparent overturning instability condition that is difficult to reconcile with observations in past earthquakes. The SEAOC Commentary suggests supplemental criteria for determining overturning to the foundations (also refer to para 4-4*a*(3) and 4-8).

c. Direction of Force

(1) *Horizontal forces.* In general, the horizontal design earthquake forces are applied nonconcurrently in the direction of each of the main axes of the structure (chap 3, para 3-3(D)). However, in some cases a more severe condition may occur when the force is applied at a horizontal direction not parallel to the main axes. For some elements of a building, the effects of concurrent motion about both principal axes should be investigated.

(a) Buildings. An independent design about each of the principal axes will generally provide adequate resistance for forces applied in any direction. Special consideration must be made at outsi corners and re-entrant corners for the vulnerable e fects of concurrent motions about both principal axes. An approved procedure for investigating the effects of concurrent motion on the vulnerable elements is to combine the seismic forces acting in the direction on one axis with 0.3 times the force effects resulting from the seismic forces acting in the direction perpendicular to the first axis.

(b) Structures other than buildings. For structures circular in plan, such as tanks, towers, and stacks, the design should be equally resistant in all directions. For four-legged structures substantially square in plan, seventy percent (70%) of the prescribed forces should be applied concurrently in the directions of the two principal axes, especially for purposes of designing for overturning effects on columns and foundations.

(2) *Vertical forces.* Vertical components of ground motion are not usually calculated but considered to be accounted for in the difference between the vertical load capacity and actual vertical loads and in special provisions using reduced dead loads. Such provisions include the 0.9 factor for dead load in chapter 6, formula 6-2, and chapter 7, formula 7-2, for considering the minimum gravity loads (chap 3, para 3-3(J)2c). These reduced loads apply t axial compression due to gravity in concrete c umns and walls when subjected to seismic bending moments and uplift forces and to beam bending moments due to gravity when combined with seismic bending moments in the opposite direction (i.e., bending moment reversal).

(a) Horizontal elements. In Seismic Zones 3 and 4, special considerations must be given to the effects of vertical accelerations on horizontal prestressed elements (especially those with draped prestressing) and horizontal cantilevers (chap 3, para 3-3(A)4). An approved procedure for investigating the effects of vertical accelerations for the horizontal prestressed elements is to rely on only fifty percent (50%) of the dead load as a minimum gravity load when applying the lateral forces. Horizontal cantilever elements should be checked for the capacity of the elements to resist a net upward force of twenty percent (20%) of the dead load.

(b) Hold-downs. In Seismic Zones 3 and 4, the design of hold-downs to resist bending moments and uplift forces will use a maximum of 0.9 of the dead load for gravity resistance.

d. Path of Forces. All of the inertia forces originating from the masses on and off the structr must be transmitted from their source to the base the structure (see fig 4-5 and 4-6).

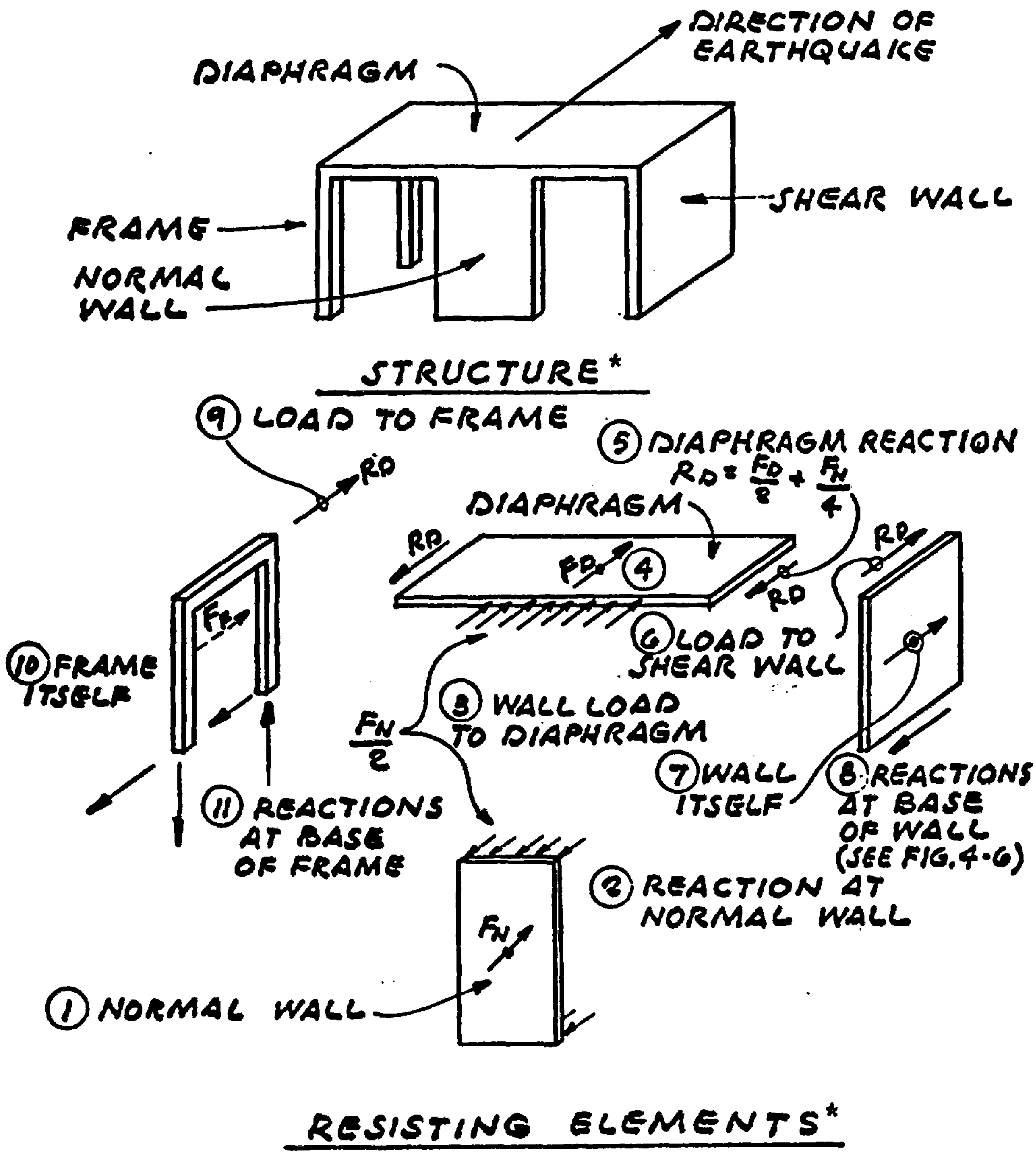


Figure 4-5 Path of forces

**Note:* Example shows flexible diaphragm. For rigid diaphragm, relative rigidities and torsion will be considered.

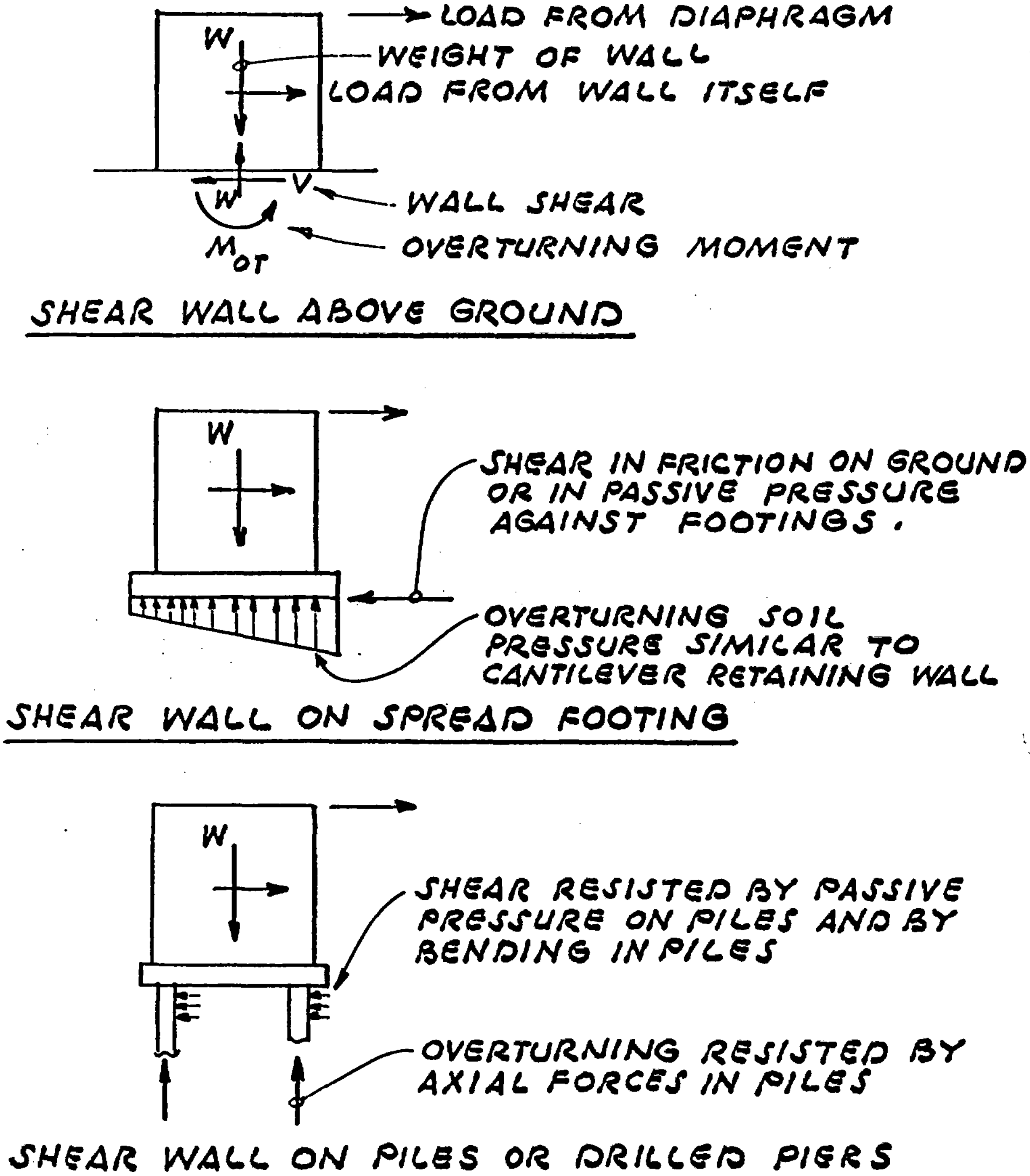


Figure 4-6 Transfer of forces to ground

(1) Forces normal to the plane of a wall must be transferred either vertically to the floors above and below or horizontally to frames or shear walls. These forces will be governed by formula 3-8.

(2) Diaphragms acting as horizontal beams must transfer inertia forces to the frames and/or shear walls. These forces will be governed by formulas 3-9 and 3-9A. In some cases, the diaphragm forces are transferred to a collector member (or a drag strut). This strut load must, in turn, be transferred to the shear wall.

(3) Frames and shear walls must transfer forces contributed from the diaphragms as well as their own inertia forces to the foundations. These forces are governed by formulas 3-1, 3-6, and 3-7.

(4) Forces applied to the foundations by the shear walls and frames must be transmitted into the ground. See paragraph 4-8 for design of foundations.

(5) Connections between all elements must be capable of transferring the applied forces from one element to another. Special design requirements for connections are reviewed in paragraph 4-6.

e. Rigidity Analysis

(1) *Horizontal forces.* For rigid diaphragms, the horizontal forces are transferred to the vertical frames and shear walls in proportion to the relative rigidities. When all the vertical elements (frames or shear walls) are of equal size in a symmetrical building, the diaphragm forces are distributed equally. When there are large differences or a lack of symmetry, a rigidity analysis must be performed. When the diaphragms are flexible, the horizontal forces are transferred in proportion to tributary area. (See chap 3, para 3-3(E)4, and chap 5, para 5-2*d*.)

(2) *Horizontal torsional moments.* For rigid diaphragms, where the center of rigidity of the vertical lateral force-resisting elements (frames or shear walls) is not coincident with the center of mass, provisions must be made for this eccentricity. For a symmetrical building, a minimum eccentricity of 5 percent of the maximum building dimension is required. (See chap 3, para 3-3(E)5, and chap 5, para 5-2*d*.)

(3) *Distribution between shear walls and frames (dual systems).* When a dual bracing system is used (table 3-3, Category 3, K = 0.80), a rigidity analysis must be made to determine the interaction between the walls and the frames. Generally for tall buildings, shear walls deflect as vertical cantilevers in a concave shape and frames deflect in a straight line or convex shape (see fig 4-7). In a dual system with rigid diaphragms, the shear walls and frames are forced to deflect the same amount at each story: therefore, some force transfer must occur between shear walls and frames. Shear walls tend to support the frame at the lower stories and the frame tends to support the shear wall at the upper stories (see fig 4-7). (Also, see chap 6, para 6-2*d*(3).)

f. Elements Not Part of the Lateral Force-Resisting System. The elements designated as the lateral force resisting-system must be designed to resist the total applied lateral force. In addition, all load-carrying elements not designed to be part of the lateral force-resisting system must be analyzed to determine if they are compatible with the lateral force-resisting system (see chap 3, para 3-3(J)1d and e). Any element that is not strong enough to resist the forces that it attracts or the interstory drifts that occur will be damaged unless it is isolated from the lateral force-resisting system.

g. Dynamic Approach. Alternative methods to the static distribution of seismic forces are permitted by chapter 3, paragraph 3-3(I). Some basic concepts are discussed in chapter 2, paragraphs 2-4 and 2-10.

4-5. Design of the structural elements. Design of diaphragms, walls, and frames are covered by chapters 5, 6, and 7, respectively. These structural elements must be designed for various combinations of loads and must satisfy certain deformation requirements.

a. Load combinations will be in accordance with chapter 3, paragraph 3-3(J)2c.

b. Structural elements will be designed to resist the combined axial, shear, and bending forces.

c. Deformations will be governed by the provisions for interstory drift (chap 3, para 3-3(H)1), building separations (chap 3, para 3-3(H)2, and para 4-7), deformation compatibility (chap 3, para 3-3(J)1d), diaphragm deformation (chap 3, para 3-3(J)2d, and chap 6, para 6-2*b*), and exterior elements (chap 3, para 3-3(J)3d).

(1) For determining compliance with the deformation provisions, only structural elements should be considered in the stiffness calculations. It is unconservative to include the stiffness participation of nonstructural elements without substantiated data. This is in contrast with the assumptions used in the period calculation for obtaining values for C and S (para 4-3*d*(4)). Thus, it is not uncommon to have one set of stiffness assumptions for calculating the total design lateral forces and another set of stiffness assumptions for calculating the design lateral displacements. It is acceptable to calculate the lateral deformations based on lateral forces corre-

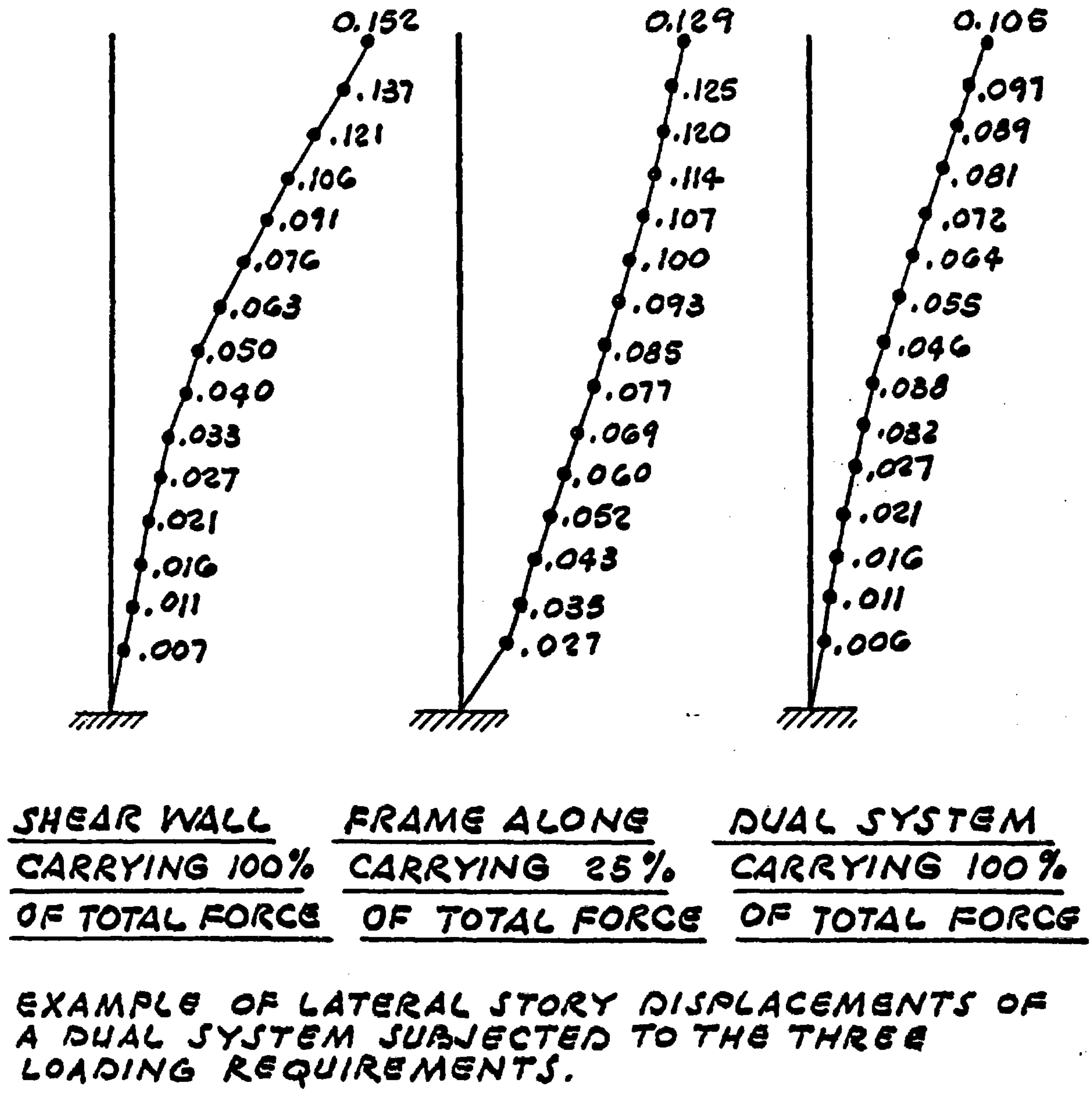


Figure 4-7 Dual system-deformations

sponding to a building period T_D longer than the period T used for the design lateral forces and without the limit specified in paragraph 4-3*d*(5). An example is given below.

(2) In the seven-story building example in table 4-4, C and S are based on a building period T of 0.8 second. The design lateral forces include an additional top story force F_t of 44 kips and a total lateral force V of 791 kips. The calculated period based on the bare structural frame is 1.2 seconds. This period is not valid for use in calculating the lateral forces because it ignores some elements that will stiffen the structure (para 4-3*d*(4)) and it exceeds the recommended maximum limit in paragraph 4-3*d*(5). However, the period of 1.2 seconds may be used as T_D to calculate the lateral forces used to determine the lateral displacement. The resulting $(F_t)_D$ is 54 kips and V_D is 644 kips. Therefore, to calculate the lateral displacement, the values of 54 kips and 644 kips may be used in lieu of 44 kips and 791 kips, respectively. This reduces the calculated displacement from 2.7 inches to 2.2 inches. This displacement will be multiplied by 1.0/K to determine drift compliance or by 3.0/K to determine deformation compliance with provisions in chapter 3.

d. The secondary effects of lateral deformation (P-Δ effect), when significant, must be investigated to assure lateral stability.

4-6. Connections between elements. Foremost among requirements vital to earthquake-resistant design of all types of buildings is the necessity of tying the various structural elements together so that they act as a unit. Possibly the most important aspect of lateral force design is the connections (seams and joints) between the structural elements. In designing and detailing, it is well to keep in mind that the lateral forces are not static, as assumed for convenience, but dynamic and to a great extent unpredictable. Since prevention of collapse during a severe earthquake depends upon the energy absorbing capacity of the structural elements, the ultimate strength of the structure should be governed by the strength of the structural elements rather than by the strength of their connections; thus, connectors should not be the weak link of the structure. Obviously, a structural element cannot transmit shears, moments, and torsions in excess of the ultimate strength of the connection used to join elements. As a general rule these connections should be sufficient to develop the useful strength of the structural elements connected, regardless of calculated stress.

a. Design Criteria. Special design requirements for connections are included in the following paragraphs of this manual.

(1) *Chapter 3, paragraph 3-3(J)1g, Braced Frames.* Connections of braced frames must be designed to develop the full tension and compression capacity of the members or they must be designed for 1.25 times the design lateral force without the usually permitted one-third increase.

(2) *Chapter 3, paragraphs 3-3(J)2d, Diaphragms; 3-3(J)3a, Anchorage of Concrete or Masonry Walls; and 3-3(J)3b, Wood Diaphragms Used to Support Concrete or Masonry Walls.* These provisions specify the minimum requirements for connecting floors and roofs to concrete and masonry walls.

(3) *Chapter 3, paragraph 3-3(J)3d, Exterior Elements.* Connections of precast or prefabricated non-bearing, non-shear wall panels or similar elements must be designed in accordance with special provisions for story drift, seismic design forces, and ductility.

(4) Chapter 5, Diaphragms; chapter 6, Walls; chapter 7, Space Frames; and chapter 8, Reinforced Masonry, provide additional minimum connection requirements for lateral force-resisting structural systems.

b. Forces. Forces to be considered in design of connections between structural elements, in addition to lateral force shears, are axial loads, flexural and torsions (twisting), as well as secondary or prying forces within connections—separately or combined as applicable to the specific case. These forces, at juncture seam along the intersection of the structural elements, may be the resultant of gravity loads, overturning, differential foundation settlements, lateral forces both normal and parallel to vertical elements, and shrinkage and thermal forces. Positive means will be provided for transferring shears from the plane of the diaphragm into the vertical resisting elements, and also for transferring wind or seismic forces from the vertical elements into the diaphragm. In designing connections or ties, it is necessary to make each and every connection consistent with the basic assumptions and distribution of forces. Provisions will be made in the design of connections to lateral force movements in walls arising from creep, temperature, and shrinkage movements in decks, including steel beams or girders when decking is fastened thereto. All significant loadings must be considered, and the joints and connections designed for forces consistent with all reasonable combinations of loadings.

c. Details. Details of connections shall admit to a rational analysis in accordance with well-established principles of mechanics. Joints and connections may be made by welding, bolting, by bond and anchorage of reinforcement, by dowels, and by mechanical devices such as embedded shapes and welded studs. The transfer of shear may be accomplished by using reinforcing steel extended as dowels coupled with cast-in-place concrete placed between roughened concrete interfaces. The entire shear should be considered as transferred through one type of device, even though a combination of devices may be available at the joint or support being considered unless one is sure that the combination of devices will act in unison. Because joints and connections directly affect the integrity of the structure, their design and fabrication must be adequate for the functions intended. Rotational forces resulting from eccentric connections must be considered. In general, elements and members should be detailed that torsion and moments are held to a minimum the connections.

d. Allowable Shear and Tension on Bolts in Concrete. Table 4-5 shows the maximum allowable forces on steel bolts (A307 or better) embedded in regular weight concrete (3,000 psi minimum strength). Values are based on a bolt spacing of at least 12 diameters with a minimum edge distance of 6 diameters. The bolts will have a standard bolt head or an equal deformity in the embedded portion. In Seismic Zone Nos. 2, 3, and 4, an additional 2 inches of embedment will be provided for anchor bolts located in the top of columns. When combining tension and shear forces on a bolt, the following interaction formula is applicable:

$$\frac{\text{Design Shear Force}}{\text{Allowable Shear Force}} + \frac{\text{Design Tension Force}}{\text{Allowable Tension Force}} < 1.0 \quad (4\text{-}2)$$

Table 4-5. Allowable Shear and Tension on Bolts in Concrete[1]

Diameter (inches)	Minimum embedment[2] (inches)	Shear (pounds)	Tension[3] (pounds)
1/2	4	2,000	950
5/8	4	3,000	1,500
3/4	5	3,500	2,250
7/8	6	4,100	3,200
1	7	4,100	3,200
1-1/8	8	4,500	3,200
1-1/4	9	5,300	3,200

[1]Minimum concrete strength is 3,000 psi.
[2]An additional 2 inches of embedment will be provided for anchor bolts at tops of columns for buildings located in Zones 2, 3, and 4.
[3]Where special inspection is provided tension values may be doubled.
Note. Adopted from Uniform Building Code, 1979 edition, by International Conference of Building Officials.

4-7. Special seismic detailing. Some of the general requirements and details for satisfactory performance under earthquake conditions are enumerated and discussed in the following paragraphs. Also, refer to chapter 2, paragraph 2-9*k*.

a. Separation of Structures (chap 3, para 3-3(H)2). In past earthquakes the mutual hammering received by buildings in close proximity to one another has caused significant damage. The simplest way to prevent damage is to provide sufficient clearance so that free motion of the two structures will result. The motion to be provided for is produced partly by the deflections of the structures themselves and partly by the rocking or settling of foundations. The gap must equal the sum of the total deflections from the base of the two buildings to the top of the lower building.

(1) In the case of a normal building, less than 80 feet in height using concrete or masonry shear walls, the gap shall be not less than the arbitrary rule of 1 inch for the first 20 feet of height above the ground plus 1/2 inch for each 10 feet of additional height.

(2) For higher or more flexible buildings, the gap or seismic joint between the structures should be based on 3/K times the deflections determined from the required (prescribed) lateral forces. If the design of the foundation is such that rotation is expected to occur at the base due to rocking or due to settlement of foundations, this additional deflection (as determined by rational methods) will be included.

b. Seismic Joints. Junctures between distinct parts of buildings, such as the intersection of a wing of a building with the main portion, are often designed with flexible joints that allow relative movement. When this is done, each part of the building must be considered as a separate structure that has its own independent bracing system. The criteria for separation of buildings in paragraph *a* above will apply to seismic joints for parts of buildings. Seismic joint coverages will be made flexible, waterproof, and architecturally acceptable.

(1) An example that is frequently found in large one-story industrial buildings with a relatively flexible frame follows:
At one end of the industrial building it is desired to provide a small office section with stiff exterior or interior walls. The office unit is relatively much stiffer than the rest of the building. If these two units are tied together, the horizontal force of the entire structure will be delivered to the small stiff unit which may be incapable of resisting such large forces (or excessive torsion may be developed in the larger structure). Extensive damage has been observed from past earthquakes which can be attributed to the omission of such separation. A separation between the two units will be required in such cases.

(2) As an alternative to integral construction or full separation, a properly substantiated separation by a mechanical acting joint designed to take appropriate forces and displacements is permitted.

c. Bridges Between Buildings. Certain types of structures commonly found in industrial installations are tied together at or near their tops by connecting parts such as piping, conveyors, ducts, etc. For instance, it may be necessary to connect two buildings by a covered bridge or passageway. In most cases it would not be economically feasible to make such a bridge sufficiently rigid to force both buildings to vibrate together. A sliding joint at one or both ends of the bridge can usually be installed. In general, it is preferable to avoid bridges between buildings in Seismic Zone Nos. 3 and 4.

d. Stairways. Stairways may be considered as inclined extensions of horizontal diaphragms. Since the stairway has a vertical component it must be considered as a vertical shear wall and designed as such or be cut loose so as not to act in the case of earthquake shock. If the stiffness of the stairway acting as an inclined vertical shear wall is relatively small when compared to other vertical resisting elements in the building, the problem becomes less important. Thus, in general, the use of concrete stairs in a stiff building with masonry or concrete walls may be satisfactory. However, more flexible steel stairs should generally be used in buildings having a flexible moment-resisting frame. Interior stairs usually create a hole in the diaphragm which should be treated as an opening in the web of a plate girder.

e. "Short-Column" Effects. Whenever the lateral deflection of any column is restrained, when full-height deflections were assumed in the analysis, it will carry a larger portion of the lateral forces than assumed. In past earthquakes, column failures have frequently been inadvertently caused by the stiffening (shortening) effect of deep spandrels, stairways, partial-height filler walls, or intermediate bracing members. Unless considered in the analysis, such stiffening effect shall be eliminated by proper detailing for adequate isolation at the juncture of the column and the resisting elements.

4-8. Design of foundations. The foundations must be designed for the seismic forces transmitted by the shear walls and frames of the lateral force-resisting system. The media used for the transmission of horizontal forces may be friction between floor slab and ground; friction between bottom of footing and ground; and/or passive resistance of earth against vertical surfaces of footings, grade beams, or basement walls. The overturning effects, which require a careful analysis of permissible overloads for combined effect of vertical and lateral loads, will be made as part of the foundation design (refer to para 4-4*b* and to the SEAOC Commentary on overturning for additional discussion of overturning effects). Resulting tensile forces must be resisted by anchorage into the foundation. Stability against overturning must be provided for the short-time loading during an earthquake (or wind) without imposing such restrictions as to create wide disparity in foundation settlements under normal loading. This disparity could create more damage to the structure than that which might occur in an earthquake under highly increased soil pressures. The soil pressure resisting combined static and prescribed seismic loads can generally exceed the normal allowable pressure for static loads by 1/3. However, the various types of soils react differently to short-time seismic loading and any increase over normal allowable static loading will be confirmed by a soils analysis. In no case will the footing size be less than that required for static loads alone. Earthquake vibrations may cause consolidation or liquefaction of loose soils, and the resultant settlement of building foundations usually will not be uniform. In the case of rigid structures supported on individual spread footings bearing on such material, excessive differential settlements can result in damage to the superstructure. Stabilization of the soil prior to construction or the use of piles, caissons, or deep piers bearing on a firm stratum may be the solution to this problem.

a. Foundation Ties. This paragraph supplements the design criteria of chapter 3, paragraph 3-3(J)3c. Individual pile, caisson, and deep pier footings of every building or structure in Seismic Zones 2, 3, and 4 will be interconnected by ties. For Seismic Zone 1, provide ties only when surrounding soil has low passive resistance values. Each tie will be designed to carry an axial tension and compression horizontal force equal to 1/10 the larger pile cap loading. Isolated spread footings on soil with a low passive resistance will also be tied together in a way to prevent relative movement of the various parts of the foundation with respect to each other. Passive resistance values vary greatly with type of soil and depth. Adequacy of passive resistance should be determined by the soils specialist. Passive resistr or lateral bearing values are permitted only wh concrete is deposited directly against natural ground or the backfill is well compacted. Passive resistance should not be used where the lateral bearing surface is close to an excavation unless such excavation is carefully backfilled with well-compacted material. The shear in the earth between such bearing surface and open or poorly compacted excavation or a similar depression may be a critical item. Where a building is supported by piles, caissons, or deep piers, it is frequently necessary to develop horizontal shear through lateral bearing against the side of the pile, pier, or caisson. The upper soils may not have sufficient lateral bearing value to resist the lateral forces. This creates bending in the piles which must be provided for in the design. Where a building is supported on piles driven through very poor material it is frequently economical to drive batter piles to take care of horizontal shear transfer to the ground. In instances where footings are subjected to lateral thrusts due to applied vertical loads, such horizontal thrust will be added to the lateral seismic force indicated above. An example of this case could be the outward thrusts on footings of a rigid gable bent due to applied vertical loads. The ties can be formed by an interconnecting grid ne work of reinforced concrete struts or structural s shapes encased in concrete. As an alternate, a reinforced concrete floor slab, doweled to walls and footings to provide restraint in all horizontal directions, may be used in lieu of the grid network of ties. Slabs-on-grade will not be used as ties when significant differential settlement is expected between footings and slab. In such cases, slabs-on-grade will be cut loose from footings and made free floating (note that the effective unsupported height of the wall is increased for this condition). Strut ties placed below such slabs shall be cushioned or separated from the slab sufficiently so that slab settlement will not damage the strut ties. Alternatively, it may be more economical to overexcavate the soil under the footings and recompact to control differential settlements and to increase passive resistance so as to eliminate need for footing ties.

b. Pile Foundations. For pile-supported structures subjected to horizontal loads, it must be decided whether the lateral load-carrying capacity of the vertical piles is adequate or whether batter piles should be used. The lateral load-carrying capacity of vertical piles is dependent on the properties of the soil; the size, length, and material of t' pile; and the pile grouping and spacing. These tors should be taken into consideration in estima ing the ability of vertical piles to withstand the horizontal loads.

4-9. Parts and elements of buildings. Parts and elements of buildings and their anchorages will be designed for forces in accordance with chapter 3, paragraph 3-3(G), formula 3-8, and table 3-4.

a. Structural elements include walls and parapets with lateral loads normal to the flat surface, diaphragms as horizontal beams (chap 3, para 3-3 (J)2d), and penthouses (chimneys and smokestacks are covered in para *c* below). These elements will be designed to resist the specified lateral forces as well as to transfer these forces to the structural system of the building through proper connections.

b. Architectural elements include partitions, ornamentation, suspended ceilings, exterior panels (chap 3, para 3-3(J)3d), and storage racks. Architectural elements are covered in chapter 9.

c. Mechanical and electrical elements, which are covered by chapter 10, include chimneys and smokestacks, as well as equipment and machinery. For rigid and rigidly attached equipment and machinery, the force factors of table 3-4 will be used; but for flexible and flexibly mounted equipment and machinery, the special provisions of chapter 10 are required. When the mechanical and electrical elements are part of the life safety system, an "I" factor of 1.5 will be used.

4-10. Structures other than buildings. This manual is primarily concerned with the design of buildings; however, provisions are also included for some structures other than buildings. When these structures are designed in accordance with formula 3-1 in chapter 3, paragraph 3-3(D), a K-value of 2.0 or 2.5 is used as specified in table 3-3. This higher value is justified by the assumption that these structures will generally have lower damping characteristics, less inelastic deformation capacity, and less redundancy than typical buildings. Procedures and guidelines for structures other than buildings are included in chapter 11.

4-11. Final design considerations. After the structural elements have been selected and analyzed, a final design check must be made to verify that the initial assumptions are correct, and whether or not the resulting structure satisfies the intent of the seismic provisions.

a. Compare Final Sizes With Initial Estimates

(1) *Weights.* Compare the final weights of the building with the weight used to determine the seismic forces. If the weight has increased significantly (say over 5%), redesign will be necessary.

(2) *Stiffness.* If the final member sizes are substantially different than the initial estimates, a reevaluation of the design will be necessary (see para (3) and (4) below). If the relative stiffnesses of the varying elements have changed significantly, the distribution of lateral forces must be re-evaluated.

(3) *Period.* If the initial period was determined by a method using structural properties and deformation characteristics, such as in formula 3-3, the initial stiffness and weight properties must be compared to the final properties of the structure. If the final period is shorter than the initial period that was used to calculate the lateral forces, a new set of forces must be calculated and applied to the structure.

(4) *Displacements.* If the final stiffness, period, or forces have changed substantially, displacements will have to be recalculated to check for compliance with the various provisions for drift and deformation.

b. Path of Forces. Upon completion of the design, a final check will be made to determine that all the inertia forces can be transmitted without instability from their source to the base of the structure. (See para 4-4*d*.)

c. Details. Check the structural details to assure that the intent of the design calculations and the seismic design detailing are properly provided for on the construction drawings.

d. Specifications. Check the specifications to assure that the intent of the design calculations, material strength assumptions, and the seismic design detailing are properly provided for in the job specifications.

CHAPTER 5
DIAPHRAGMS

5-1. Purpose and scope. This chapter prescribes the criteria for the design of horizontal diaphragms and horizontal bracing of buildings in seismic areas, indicates principles and factors governing the horizontal distribution of lateral forces and resistance to lateral forces, gives certain design data, and illustrates typical details of construction. Refer to chapter 3, paragraph 3-3(J)2d, for design forces.

5-2. General. Buildings are composed of vertical and horizontal structural elements which resist lateral forces. Horizontal forces on a structure produced by seismic ground motion originate at the centroid of the mass of the building elements and are proportional to the masses of these elements. The forces originating at masses tributary to the horizontal elements are distributed by such horizontal elements to vertical elements which in turn transmit such forces to the ground. Forces may also be transmitted from vertical elements to horizontal elements and then be redistributed to other vertical elements. Refer to chapter 4, figures 4-4 and 4-5, for tributary weights and path of forces, respectively.

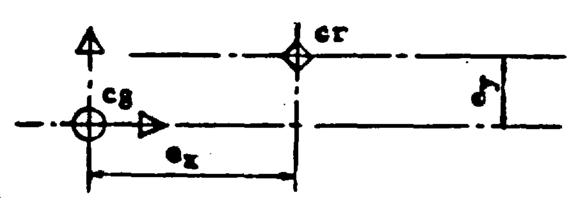
a. Function. Horizontal forces at any floor or roof level are distributed to the vertical resisting elements by using the strength and rigidity of the floor or roof deck to act as a diaphragm. Horizontal bracing may be used to act as a diaphragm to transfer the horizontal forces to the vertical resisting elements.

(1) *Diaphragms.* A diaphragm may be considered analogous to a plate girder laid in a horizontal (or inclined in the case of a roof) plane where the floor or roof deck performs the function of the plate girder web, the joists or beams function as web stiffeners, and the peripheral beams or integral reinforcement function as flanges (fig 5-1, 5-2, and 5-3). A diaphragm may be constructed of materials such as concrete, wood, or metal in various forms. Combinations of materials are possible. Strength criteria for such materials as cast-in-place reinforced concrete and structural steel are well established and present no problem to the designer once the loading and reaction system is known. Other materials frequently used to support vertical loads in floors or roofs have well-established vertical load characteristics but have required tests to demonstrate their ability to resist lateral forces. Various types of wood sheathing and steel decks fall in this category. Where a diaphragm is made up of units such as plywood, precast concrete floor units or steel deck units, its characteristics are, to a large degree, dependent upon the attachments of one unit to another and to the supporting members.

(2) *Horizontal bracing system.* A horizontal bracing system may be of any approved material, such as reinforced concrete, structural steel or wood. The bracing system will be fully developed in both directions so that the bracing diagonals and chord members form complete horizontal trusses between vertical resisting elements (fig 5-4). Deflections and web flexibility due to the required static forces will be determined using normal design principles. The stiffness category and span/depth limitations that apply to diaphragms (see para *d*, *e*, and *f* below) also apply to horizontal bracing systems. The general layout of a bracing system and sizing of members must be determined for each individual case.

b. Symbols and Notations. Additional terminology which relates to diaphragms and which will be used in this chapter is shown below:

NS = North-South direction
EW = East-West direction
e = Distance between center of gravity (CG) of forces and center of rigidity (cr) of the vertical resisting elements
RR = Relative rigidity
V = Shear (or reaction)
Δ_v = Deflection of vertical element
Δ_d = Deflection of diaphragm



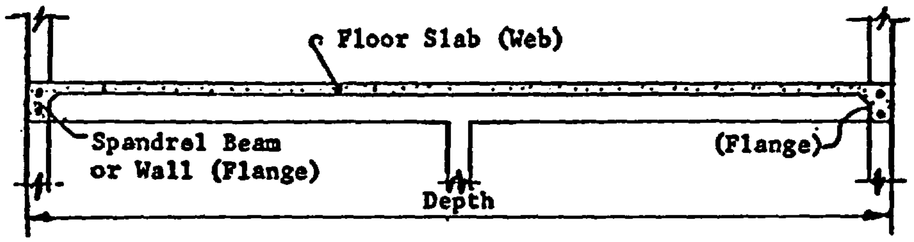


Figure 5-1. Floor Slab Diaphragm

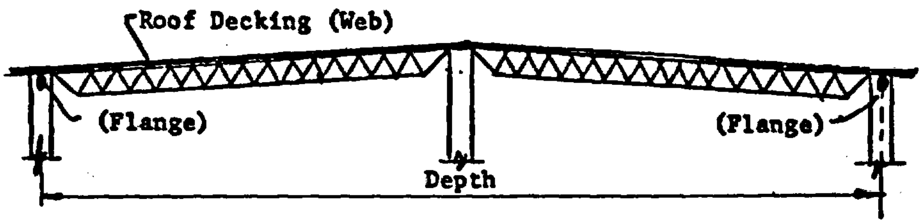


Figure 5-2. Roof Deck Diaphragm

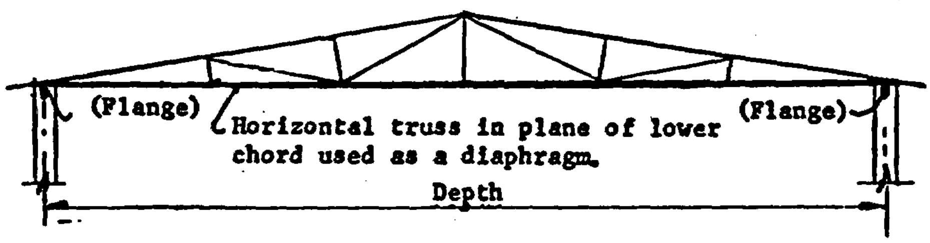
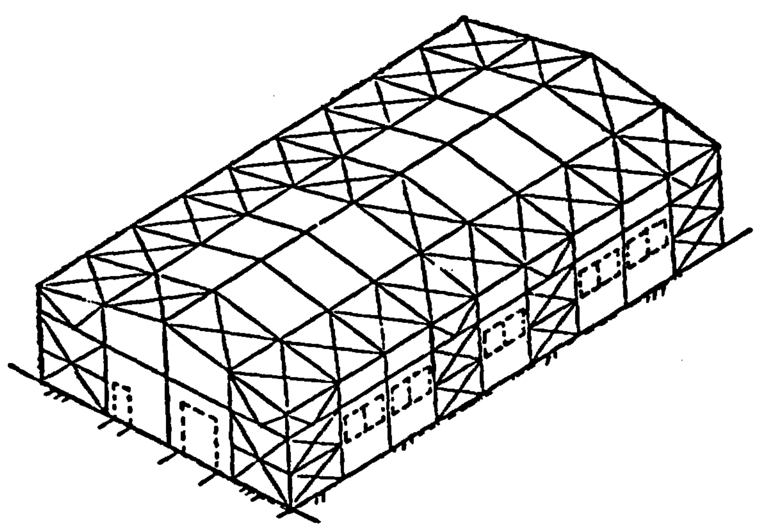


Figure 5-3. Truss Diaphragm



Upper chord shown as truss diaphragm. The truss diaphragm may also be in lower chord as shown in Figure 5-3.

Figure 5-4. Bracing an Industrial Building *c. Seismic Loadings.* Floors and roofs used as diaphragms will be designed for lateral forces specified in chapter 3, paragraph 3-3(J)2d, acting in any horizontal direction. These forces include inertia forces originating from the weight of the diaphragm and the elements attached thereto, as well as forces that are required to be transferred to vertical resisting elements because of offsets or changes of stiffness in vertical resisting elements above and below the diaphragm (chap 4, fig 4-5 and 4-6).

d. Distribution of Seismic Forces. The total shear, which includes the forces contributed through the diaphragm as well as the forces contributed from the vertical resisting elements above the diaphragm, at any level will be distributed to the various vertical elements of the lateral force resisting system (shear walls or moment resisting frames) in proportion to their rigidities considering the rigidity of the diaphragm. The effect of diaphragm stiffness on the distribution of lateral forces is discussed and schematically illustrated below (fig 5-5). For this purpose, diaphragms are classified into five groups of flexibilities relative to the flexibilities of the walls. These are rigid, semi-rigid, semi-flexible, flexible, and very flexible diaphragms. No diaphragm is actually infinitely rigid and no diaphragm capable of carrying a load is infinitely flexible.

(1) A rigid diaphragm is assumed to distribute horizontal forces to the vertical resisting elements in proportion to their relative rigidities. In other words, under symmetrical loading a rigid diaphragm will cause each vertical element to deflect an equal amount with the result that a vertical element with a high relative rigidity will resist a greater proportion of the lateral force than an element with a lower rigidity factor (fig 5-5(b)).

(2) A flexible diaphragm and a very flexible diaphragm are analogous to a shear deflecting continuous beam or series of beams spanning between supports. The supports are considered non-yielding, as the relative stiffness of the vertical resisting elements compared to that of the diaphragm is great. Thus a flexible diaphragm will be considered to distribute the lateral forces to the vertical resisting elements on a tributary load basis. A flexible diaphragm will not be considered capable of distributing torsional stresses resulting from concrete or masonry masses (fig 5-5(d)).

(3) Semi-rigid and semi-flexible diaphragms are those which have significant deflection under load but which also have sufficient stiffness to distribute a portion of their load to vertical elements in proportion to the rigidities of the vertical resisting elements. The action is analogous to a continuous concrete beam system of appreciable stiffness on yielding supports. The support reactions are dependent on the relative stiffnesses of both diaphragm and vertical elements. A rigorous analysis is sometimes very time consuming and frequently unjustified by the results; at best, the results are no better than the assumptions that must be made. In such cases a design based on reasonable limits may be used; however, the calculations must reasonably bracket the likely range of reactions and deflections (fig 5-5(c)).

(4) Torsional moment is generated whenever the center of gravity (cg) of the lateral forces fails to coincide with the center of rigidity (cr) of the vertical resisting elements, providing the diaphragm is sufficiently rigid to transfer torsion. The magnitude of the torsional moment that is required to be distributed to the vertical resisting elements by a diaphragm is determined by the larger of the following: (a) the sum of the moments created by the physical eccentricity of the translational forces at the level of the diaphragm from the center of rigidity of the resisting elements ($M_T = F_p e$, where e = distance between cg and cr) or (b) the sum of the moments created by an "accidental" torsion of 5%. The "accidental" torsion is an arbitrary code requirement equivalent to the story shear acting with an eccentricity of not less than 5% of the maximum building dimension at that level (chap 3, para 3-3(E)5). The torsional moments will be distributed through rigid diaphragms to the vertical resisting elements in a method analogous to the torsion formula $\tau = Tc/J$ (fig 5-6). Thus the torsional shear forces can be expressed by the formula $F_T = M_T kd/\Sigma kd^2$, where k is the stiffness of the vertical resisting elements, d is the distance from the center of rigidity, and Σkd^2 represents the polar moment of inertia (*Note:* $M_T = \Sigma F_T d$). The torsional shears will be combined with the direct (translational) shears (fig 5-6(b)). However, when the torsional shears are opposite in direction to the direct shears, the lateral forces shall not be decreased. A properly evaluated and rational alternative (e.g., computer techniques) to this approach can be used (refer to SEAOC Commentary on horizontal torsional moments). When diaphragms are flexible, relative to the vertical resisting elements (e.g., wood floor diaphragms and concrete or masonry shear walls), it will be assumed that the diaphragms cannot transmit torsional moments, thus there will be no torsional distribution. Cantilever diaphragms on the other hand will distribute translational forces to vertical resisting elements, even if the diaphragm is flexible. In this case, the diaphragm and its chord act as a flexural

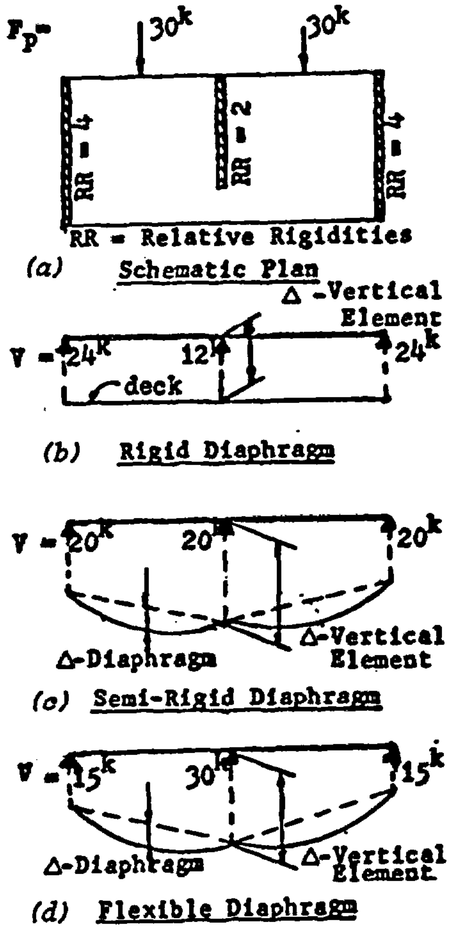


Figure 5-5. Diaphragm Flexibilities Relative to Flexibilities of the Walls

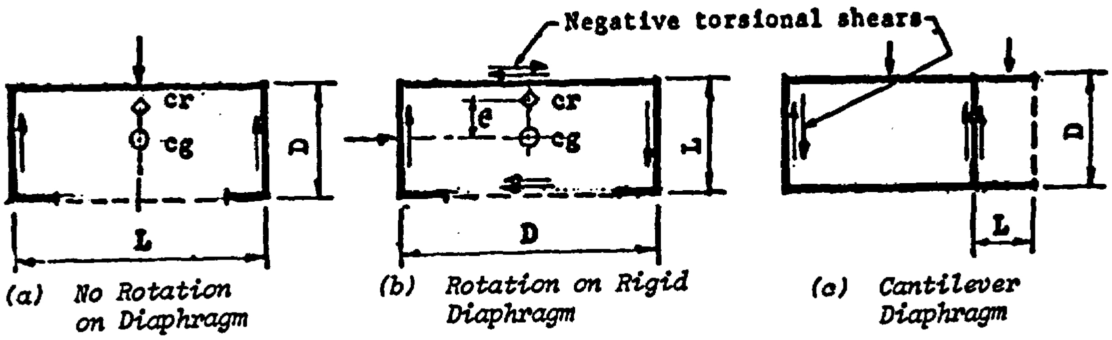


Figure 5-6. Torsional Moments on Diaphragms

beam supported by the vertical resisting elements (fig 5-6(c)).

e. Diaphragm Deflections. A diaphragm will be designed to provide such stiffness and strength so that walls and other vertical elements laterally supported by the diaphragm can safely sustain the stresses induced by the response to seismic motion. The total computed deflection (Δ_d) of diaphragms under the prescribed static seismic forces consists of the sum of two components. The first component is the flexural deflection (Δ_f) of the diaphragm which is determined in the same manner as the deflection of beams. The assumption that flexural stresses on the diaphragm web are neglected will be used except for reinforced concrete slabs. For such slabs the proportional flexural stresses also may be assumed to be carried by the web. The second component is the web (shear) deflection (Δ_w) of the diaphragm. The specific nature of the web deflection will vary depending on the type of diaphragm. The total deflection of the diaphragm under the prescribed static forces will be used as the criteria for the adequacy of the stiffness of a diaphragm. The limitation on deflection is the allowable amount prescribed for the relative deflection (drift) of the walls between the level of the diaphragm and the floor below. Refer to chap 6, fig 6-2 and para 6-2*b*. The limitation imposed on diaphragms supporting flexible walls is a maximum span-to-depth ratio, see table 5-1.

f. Flexibility Limitations. The determination and limitations of the deflections of a diaphragm is a design function. The deflections of some diaphragms can be computed with reasonable accuracy. However, other diaphragms have characteristic and fabrication variables making an accurate solution of deflection characteristics meaningless. Thus the methods of determination of the deflection characteristics for diaphragms of all materials given herein will be used to keep the range of diaphragm deflections within reasonable limits.

(1) *F-factor.* In order to provide a means of properly classifying and identifying the stiffness of a diaphragm web, the factor "F" will be introduced. The factor F is equal to the average deflection in

TABLE 5-1. Flexibility Limitation on Diaphragms

			Span/Depth Limitations			
Flexibility category	F	Maximum Span (feet)	No torsion considered in diaphragm[3]		Torsion considered in diaphragm[2]	
			Brittle walls[1]	Flexible walls	Brittle walls[1]	Flexible walls
Very flexible[4]	Over 150	50	Not to be used	2:1	Not to be used	1-1/2:1
Flexible	70-150	100	2:1	3:1	Not to be used	2:1
Semi-flexible	10-70	200	2-1/2:1	4:1	Not to be used	2-1/2:1
Semi-rigid	1-10	300	3:1	5:1	2:1	3:1
Rigid	Less than 1	400	Deflection reqm't only	No limitation	Deflection reqm't only	3-1/2:1

Notes:
[1] Walls in concrete and unit-masonry are classified as brittle; in all cases, check allowable drift before selecting type of diaphragm.
[2] When applying these limitations to cantilever diaphragms, the span/depth ratio shall be limited to one-half that shown.
[3] No torsion in diaphragm other than the 5% "accidental" torsion required by chapter 3, paragraph 3-3(E)5.
[4] For Zones 1 and 2, diagonally sheathed and plywood diaphragms in the "Very Flexible" category may be used to support laterally masonry and concrete walls in one-story buildings where the diaphragm is not required to act in rotation.

micro inches (millionths of an inch) of the diaphragm web per foot of span stressed with a shear of one pound per foot. Expressed as a formula this becomes:

$$F = \frac{\Delta_w \times 10^6}{q_{ave} L_1} \text{ where} \qquad (5\text{-}1)$$

L_1 = Distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined

q_{ave} = Average shear in diaphragm in pounds per foot over length L_1

Δ_w = Web component of Δ_d

Using the factor F, the flexibility categories of diaphragm webs have designated values as prescribed in table 5-1. The span-depth limitations do not directly reflect deflections. The web deflection will be determined by the equation

$$\Delta_w = \frac{q_{ave} L_1 F}{10^6} \qquad (5\text{-}2)$$

(2) *Determination of F-factors.* The equations for use in determining the strength and stiffness capabilities of various diaphragm materials have in most cases only been published in the literature of the companies supplying these materials. These have been based usually on a limited number of tests and have been derived empirically to fit the test data available to them. As more and more tests were run, these equations were altered to incorporate the new data. This led to many somewhat similar equations for identical diaphragm components supplied by different manufacturers. The equations used in this manual have been developed using as a basis all of the test data made available to the Triservice Seismic Design Committee at the time of the last edition of this manual (April 1973) and may be subject to some revision in the future as new data are obtained.

5-3. Diaphragm selection. In most buildings it is economical to use the roof and floor systems as diaphragms; therefore, the overall structural system, including the vertical load resisting elements, affects the selection of the diaphragm (or horizontal bracing) system. The selected system must be compatible with the criteria governing the vertical load-carrying capacities and the fire resistant qualities. Relative costs of various types of suitable diaphragms should be investigated to achieve the greatest economy. Some of the most common items that affect the selection of the diaphragm system are summarized below.

a. Transverse Frames and Longitudinal Walls or Braced Frames. For buildings such as large v houses with long span vertical moment resisting frames in the transverse direction, the diaphragm connecting these frames need be only nominal sway bracing with little or no computed stresses, since each bent would be designed to carry its tributary lateral force. However, in the longitudinal direction where only the exterior walls resist seismic forces, the diaphragm must span from side wall to side wall. If the frames are of structural steel, consideration should be given to the selection of a horizontal steel bracing system as a diaphragm. If the frames are of reinforced concrete, a concrete deck will normally be used. When applicable, torsion will be considered (para 5-2*d*(4)).

b. Multi-Story Frame Structures. For tall, multistory buildings with moment resisting frames, diaphragms will be rigid enough to distribute horizontal forces and torsion in proportion to the relative rigidities of the frames. A more flexible diaphragm on such structures must be avoided because it will permit portions of a building to vibrate out-of-phase with the rest of the structure, creating reverse warping strains.

c. Deep Beam Analogy. Diaphragms are designed as deep beams so that the web (decking sheathing) will carry shear and the flanges (spa beams or other members) at the edges will resist the bending moments. Webs of precast concrete units or metal deck units will require details for joining the units to each other and to their supports so as to distribute shear forces. Boundary members at edges of diaphragms must be designed to resist direct tensile or compressive (chord) stresses, including adequate splices at points of discontinuity. For instance, in a steel frame building the spandrel beams acting as a diaphragm flange component require a splice design at the columns for the tensile and compressive stresses induced by diaphragm action.

(1) *Openings.* Diaphragms with openings, such as cut-out areas for stairs or elevators, will be analyzed similarly to a plate girder with a hole in the web, and require complete detailing to show that all the stresses around the opening will be developed.

(2) *L- and T-shaped buildings.* The L- and T-shaped buildings will have the flange (chord) stresses developed through or into the heel of the L or T. This is analogous to a girder with a deep haunch.

d. Braced Frame Systems. When plannir a bracing system of a building, consider the stn as a whole. Visualize the ways in which the structure might fail and provide bracing with adequate strength and rigidity to keep the structure upright. Before deciding upon the position of bracing, the structural engineer must be certain just where every obstruction and other controlling features will be located (see para 5-8). (Refer to chap 6, para 6-2*d*, for vertical bracing.)

e. Connections. Connections and anchorages between the diaphragms and the vertical resisting elements will be designed to conform to chapter 3, paragraphs 3-3(J)1g, 2d, 3, and chapter 4, paragraphs 4-4*d*(5) and 4-6.

5-4. Concrete diaphragms. *a. General Design Criteria.* The criteria used to design concrete diaphragms will be ACI 318-77 (except appendix A) as modified by this paragraph. Concrete diaphragm webs will be designed as concrete slabs which may be designed to support vertical loads between the framing members or rest on other vertical load-carrying elements such as precast concrete elements or steel decks. If shear is transferred from the diaphragm web to the framing members through steel deck fastenings, the design will conform to the requirements in paragraph 5-6, Steel Deck Diaphragms.

b. Span and Anchorage Requirements. The following provisions are intended to prevent diaphragm buckling.

(1) *General.* Where reinforced concrete slabs are used as diaphragms to transfer lateral forces, the clear distance (L_v) between framing members or mechanical anchors shall not exceed 38 times the total thickness of the slab (t).

(2) *Cast-in-place concrete slabs not monolithic with supporting framing.* When concrete slabs are not monolithic with the supporting framing members (e.g., slabs on steel beams) the slab will be anchored by mechanical means at intervals not exceeding four feet on center along the length of the supporting member. This anchorage is not a computed item and should be similar to that shown on figure 5-7. For composite beams, anchorages provided in accordance with AISC provisions for composite construction will meet the requirements of this paragraph.

(3) *Cast-in-place concrete diaphragms vertically supported by precast concrete slab units.* If the slab is not supporting vertical loads but is supported by other vertical load-carrying elements, mechanical anchorages will be provided at intervals not exceeding 38_t. Thus, the provisions of (1) above will be satisfied by defining L_v as the distance between the mechanical anchorages between the diaphragm slab and the vertical load-carrying members. This mechanical anchorage can be provided by steel inserts or reinforcement, by bonded cast-in-place concrete lugs, or by bonded roughened surface, as shown on figure 5-8. Positive anchorage between cast-in-place concrete and the precast deck must be provided to transmit the lateral forces generated from the weights of the precast units to the cast-in-place concrete diaphragm and then to the main lateral force resisting system.

(4) *Precast concrete slab units.* If precast units are continuously bonded together as shown on figure 5-9, they may be considered concrete diaphragms and designed accordingly as described hereinbefore. Intermittently bonded precast units or precast units with grouted shear keys will not be used as a diaphragm.

> EXCEPTION: In Seismic Zone 1 (fig 5-9*a*), the use of hollow core planks with grouted shear keys is permitted. Also the use of connectors, in lieu of continuous bonding, for precast concrete members is permitted if the following considerations and requirements are satisfied:
> (a) Conformance with Prestressed Concrete Institute (PCI)—Design Handbook seismic design requirements.
> (b) Shear forces for diaphragm action can be effectively transmitted through the connectors. The shear will be uniformly distributed throughout the depth or length of the diaphragm with reasonably spaced connectors rather than with a few which will have localized concentration of shear stresses.
> (c) Connectors will be designed for at least two times the actual shear force.
> (d) Detail structural calculations be made including the localized effects in concrete slabs attributed from these connectors.
> (e) Sufficient details of connectors and embedded anchorage be provided to preclude construction deficiency.

(5) *Metal formed deck.* Concrete slabs that are cast by use of metal formed deck shall be governed by either the requirements of paragraphs (2) above, or the requirements of paragraph 5-6*d*, Deck with Concrete Fill, depending on the characteristics of the metal formed deck.

c. Special Reinforcement. Special diagonal reinforcement will be placed in corners of diaphragms as indicated in figure 5-10. Typical chord reinforcement and connection details are shown in figure 5-11.

d. Flexibility Factor. The web stiffness factor F

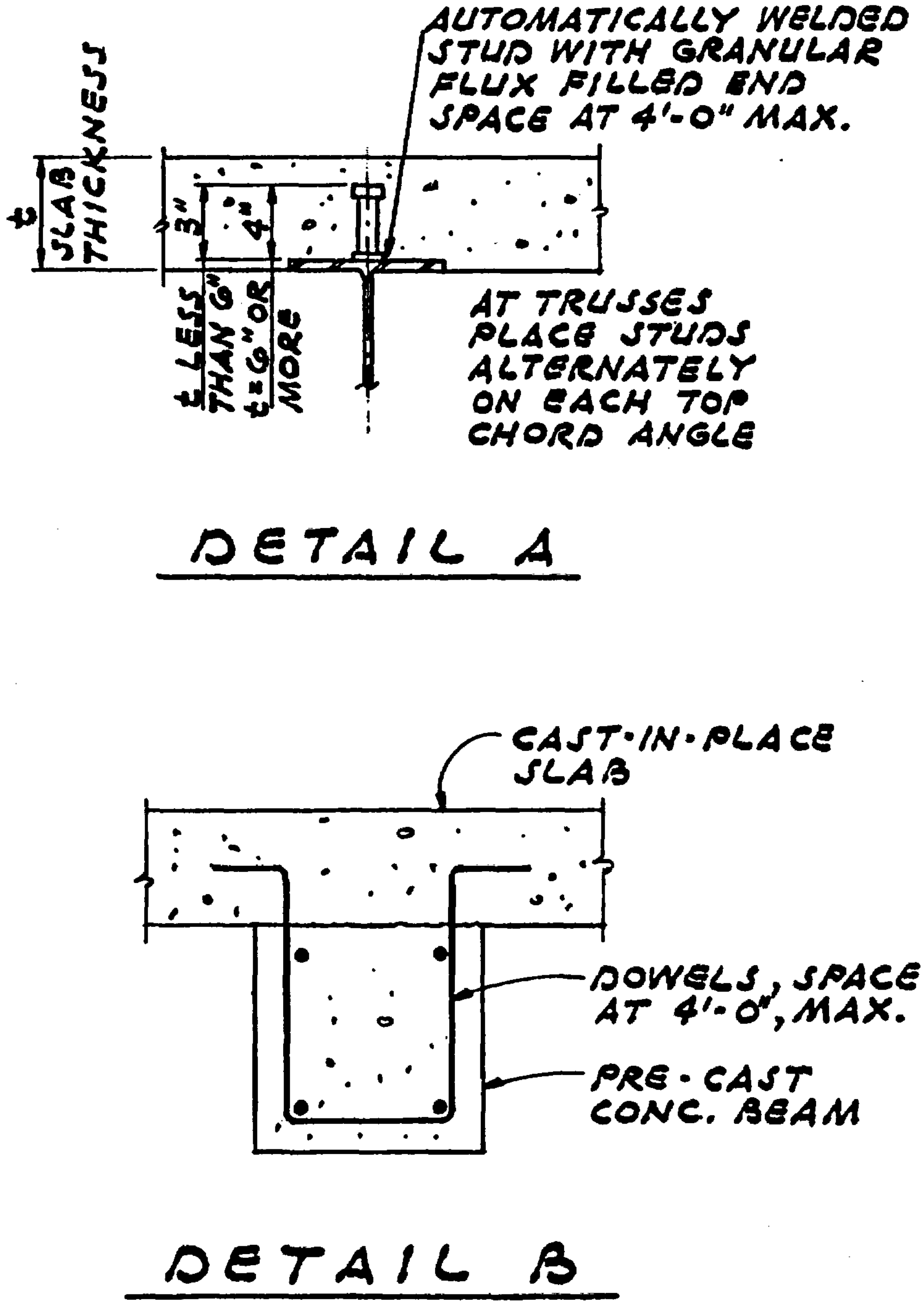


Figure 5-7. Anchorage of Cast-in-Place Concrete Slabs Not Monolithic with Supporting Framing

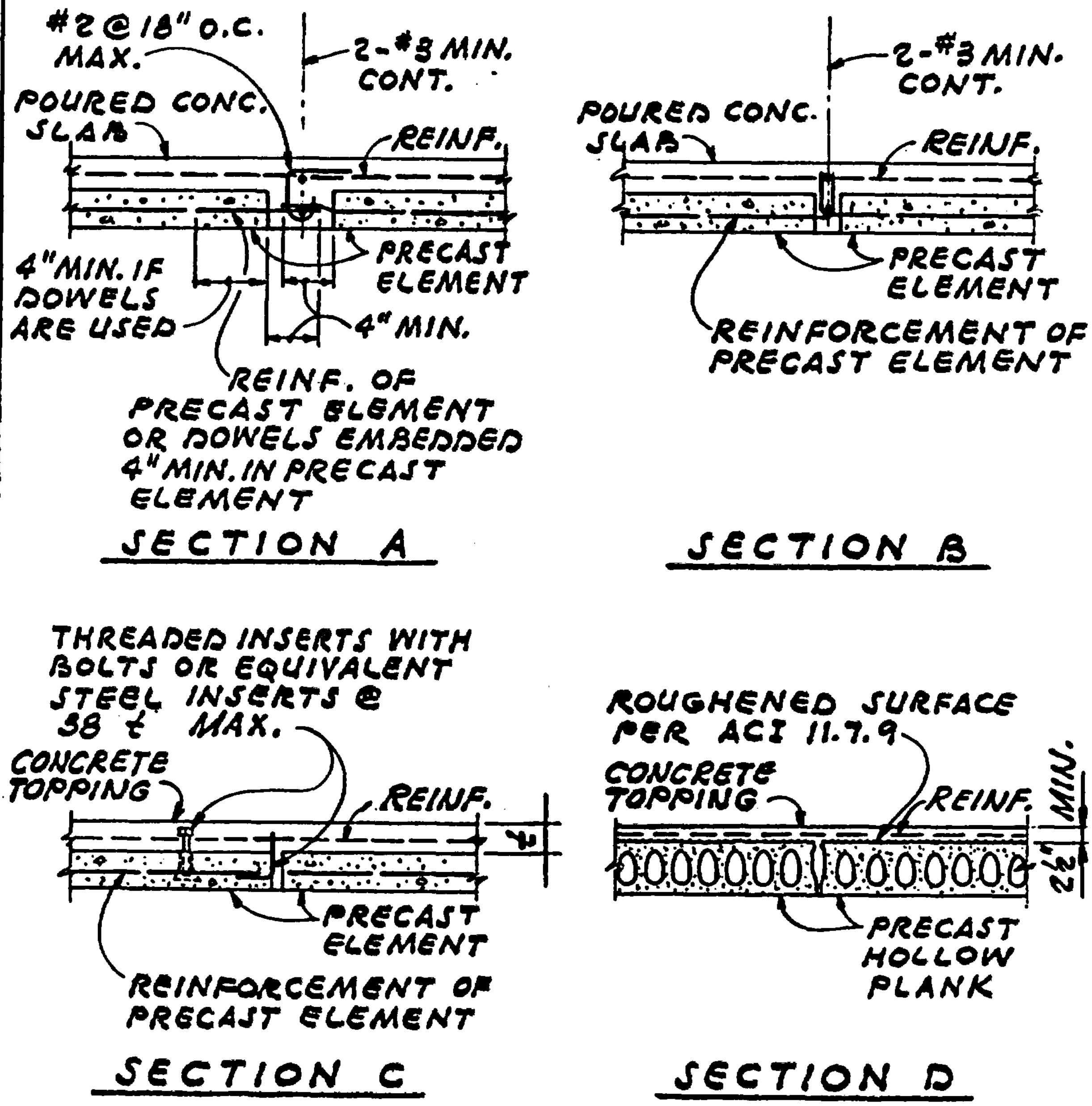


Figure 5-8. Attachment of Superimposed Diaphragm Slab to Precast Slab Units

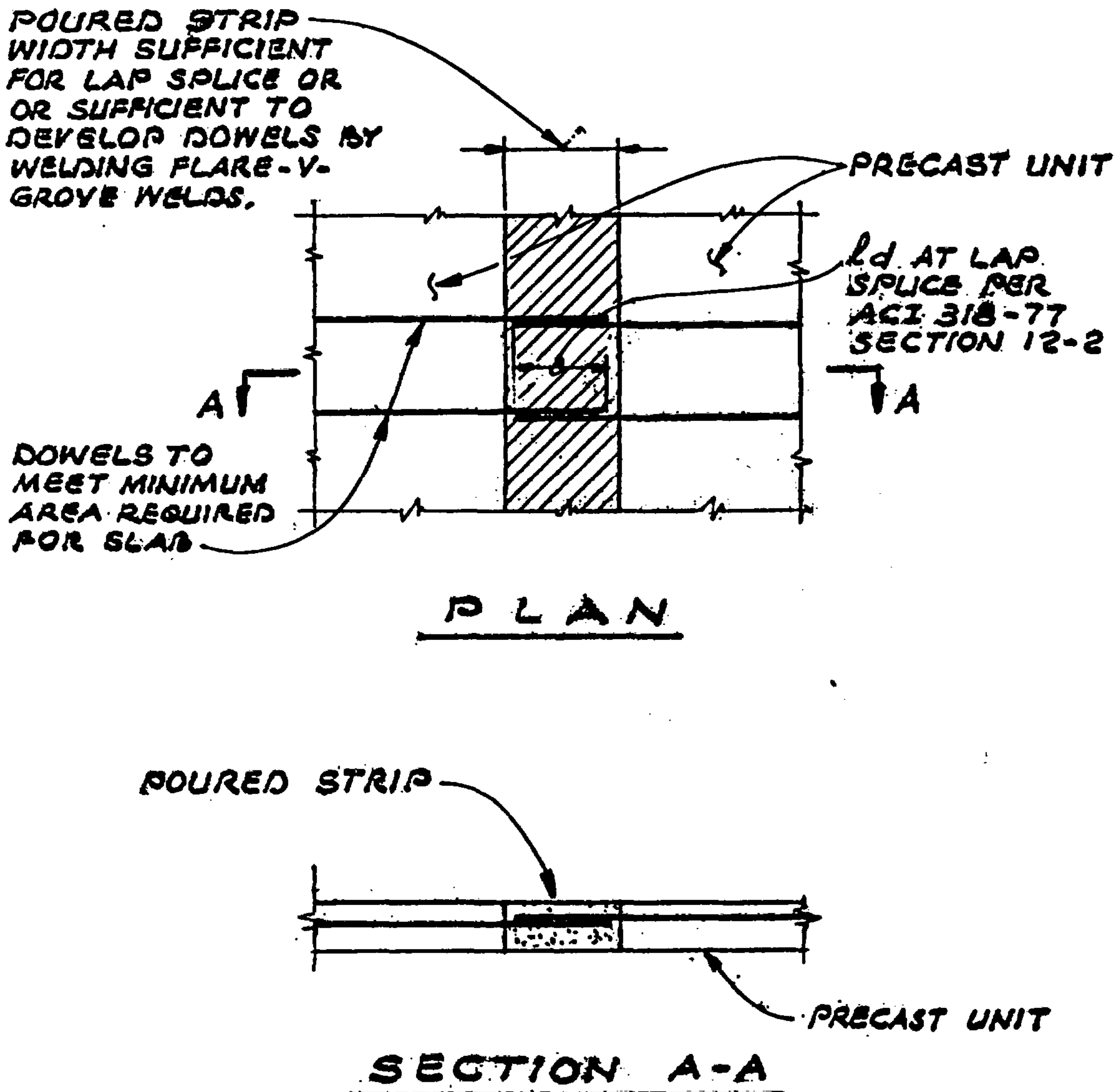


Figure 5-9. Precast Concrete Diaphragms - Requirements for Continuously Bonded Precast Unit

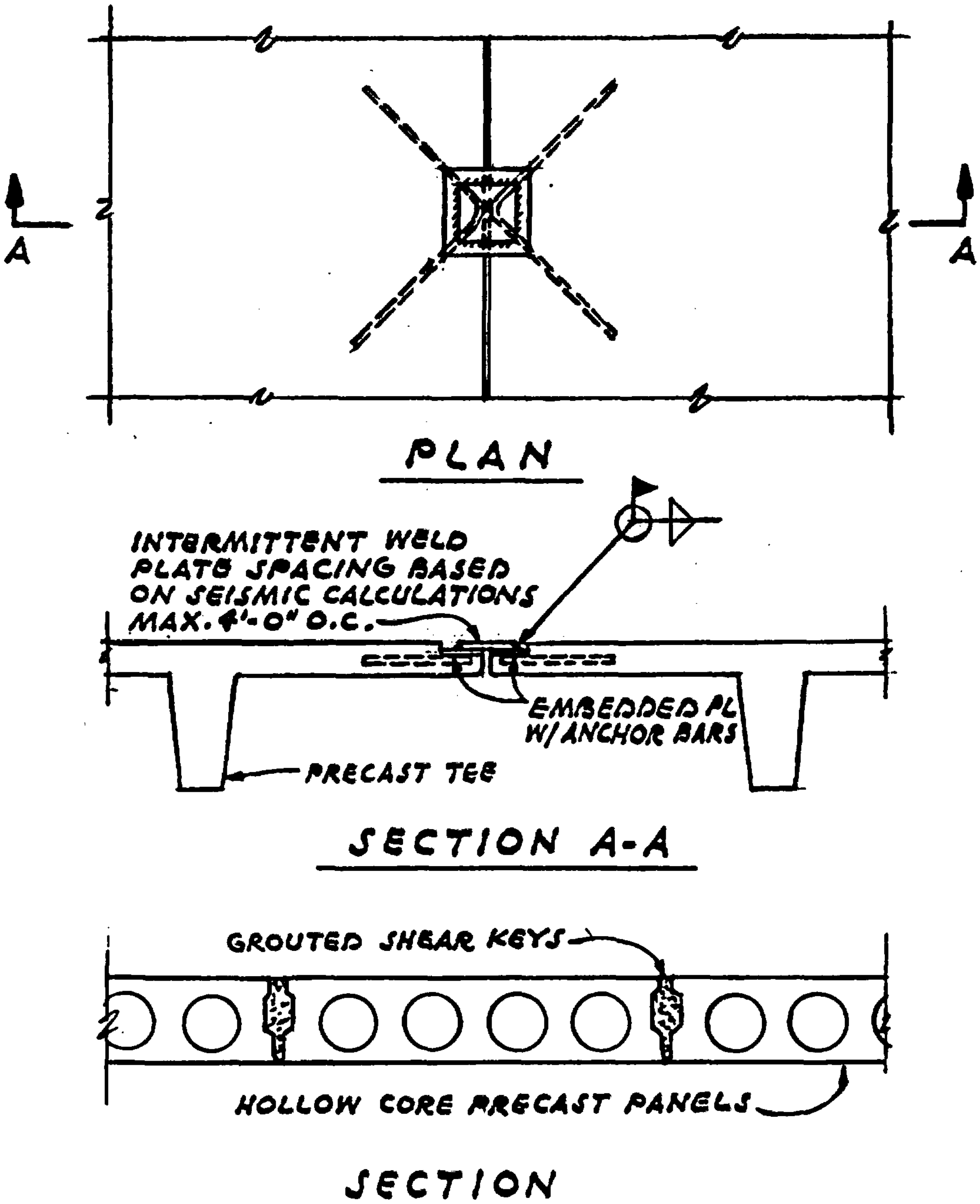


Figure 5-9a. Concrete Diaphragms Using Precast Units - Details Permitted in Seismic Zone 1 Only

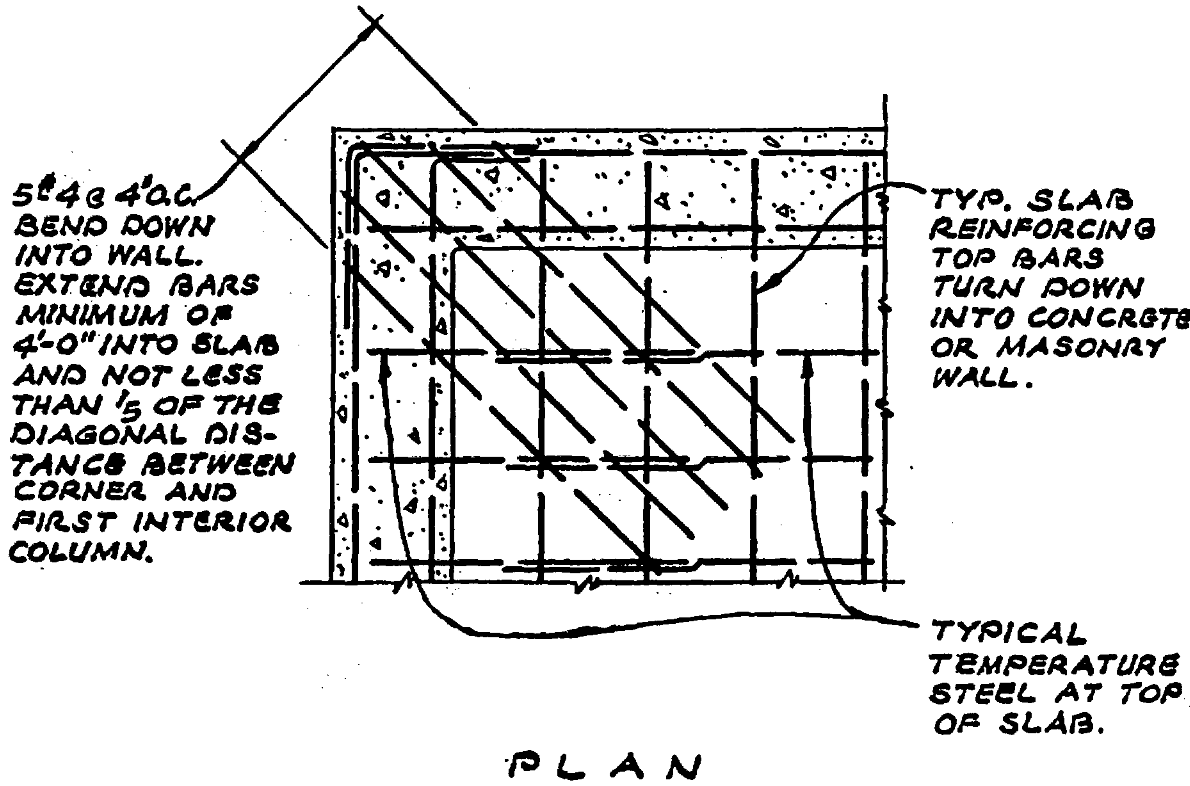


Figure 5-10. Corner of Monolithic Concrete Diaphragm

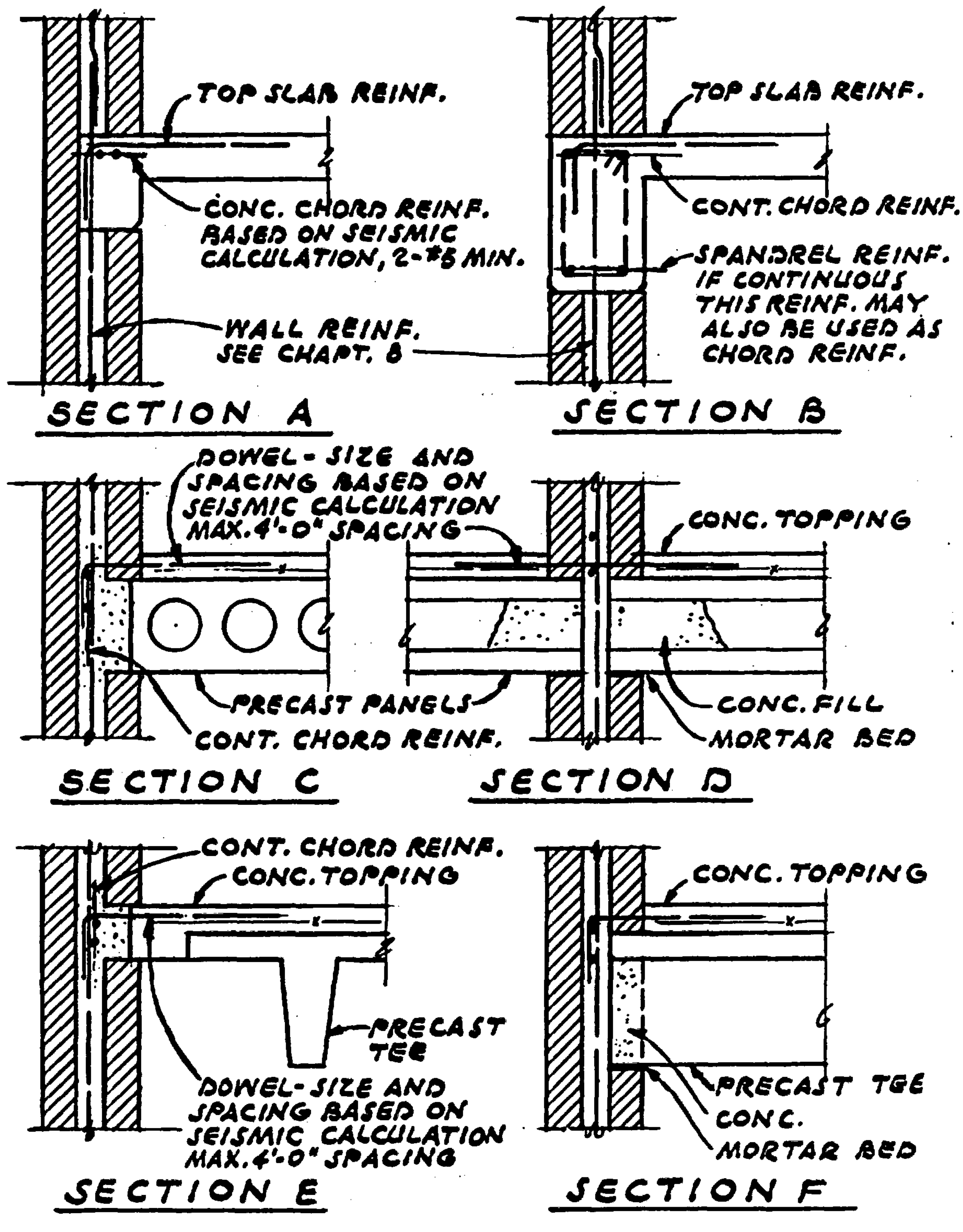


Figure 5-11. Concrete Diaphragms - Typical Connection Details

(see para 5-2*f*) will be determined by the following formula:

$$F = \frac{10^6}{8.5tw^{1.5}\sqrt{f'_c}} \qquad (5\text{-}3)$$

where:

t = Thickness of the slab in inches

w = The weight of the concrete in pounds per cubic foot. Minimum value of w will be 90 pounds per cubic foot.

f'_c = The compressive strength of the concrete at 28 days in pounds per square inch.

Diaphragms of this type are in the rigid category of stiffness and are usually limited only by the appropriate deflection limitations. The deflections of this type of diaphragm will be determined using the unfactored loads specified in chapter 3, paragraph 3-3, when controlled by the limits indicated in paragraphs 5-2*e* and *f*.

e. Electrical Raceways. The placement of electrical raceways in concrete topping slabs may result in the slab being ineffective as a diaphragm. The effect of the loss of concrete section will be considered. Coordination of structural diaphragm slab with electrical plans will be provided.

5-5. Gypsum diaphragms, cast-in-place.

a. General Design Criteria. The following criteria will be used to design cast-in-place gypsum diaphragms.

b. Shear Capacity

(1) The allowable diaphragm shear on poured gypsum concrete diaphragms will be as shown in tables 5-2 through 5-4 for roof systems using s purlins and electrically welded wire mesh.

(2) In lieu of tables 5-2 through 5-4, the following formula will be used to determine the allowable shear of the diaphragm.

$$q_D = [.16f_g tC_1 + 1{,}000(k_1d_1 + k_2d_2)]C_2 \qquad (5\text{-}4)$$

where

q_D = Allowable maximum shear per foot on diaphragm in pounds per linear foot. The one- third increase usually permitted to working stresses in seismic design is not applicable.

f_g = Oven-dry compressive strength of gypsum in p.s.i. as determined by tests conforming to ASTM C472-73.

C_1 = 1.0 for Class A gypsum concrete; 1.5 for Class B gypsum concrete.

C_2 = 1.4 for Class A gypsum concrete; 1.0 for Class B gypsum concrete.

t = Thickness of gypsum between subpurlins in inches.

k_1 = Number of mesh wires per foot passing over subpurlins.

d_1 = Diameter of mesh wires passing over subpurlins in inches.

k_2 = Number of mesh wires per foot parallel to subpurlins.

d_2 = Diameter in inches of mesh wires parallel to subpurlins.

c. Flexibility Factor. The factor F (para 5-2*e* and *f*) for determination of diaphragm stiffness and deflections will be determined by the formula

$$F = \frac{140}{\sqrt{q_D}}$$

where

q_D = The allowable shear specified in tables 5-2 through 5-4 or Formula 5-4 in pounds per foot.

Table 5-2. Shear Values of Poured Gypsum Diaphragms

Class	Compressive Strength	Poured Gypsum Thickness	Mesh	*ALLOWABLE SHEAR VALUES (q_D)	
				Bulb Tees	Trussed Tees
A	500	2½"	4 x 8 / 12 - 14	Not Allowed	890
A	500	2½"	6 x 6 / 10 - 10	Not Allowed	1,040
B	1,000	2½"	4 x 8 / 12 - 14	1,040	1,040
B	1,000	2½"	6 x 6 / 10 - 10	1,140	1,140

NOTE: *1/3 increase usually permitted on working stresses in seismic design not applicable.

*Table 5-3. Shear on Anchor Bolts and Dowels - Reinforced Gypsum Concrete**

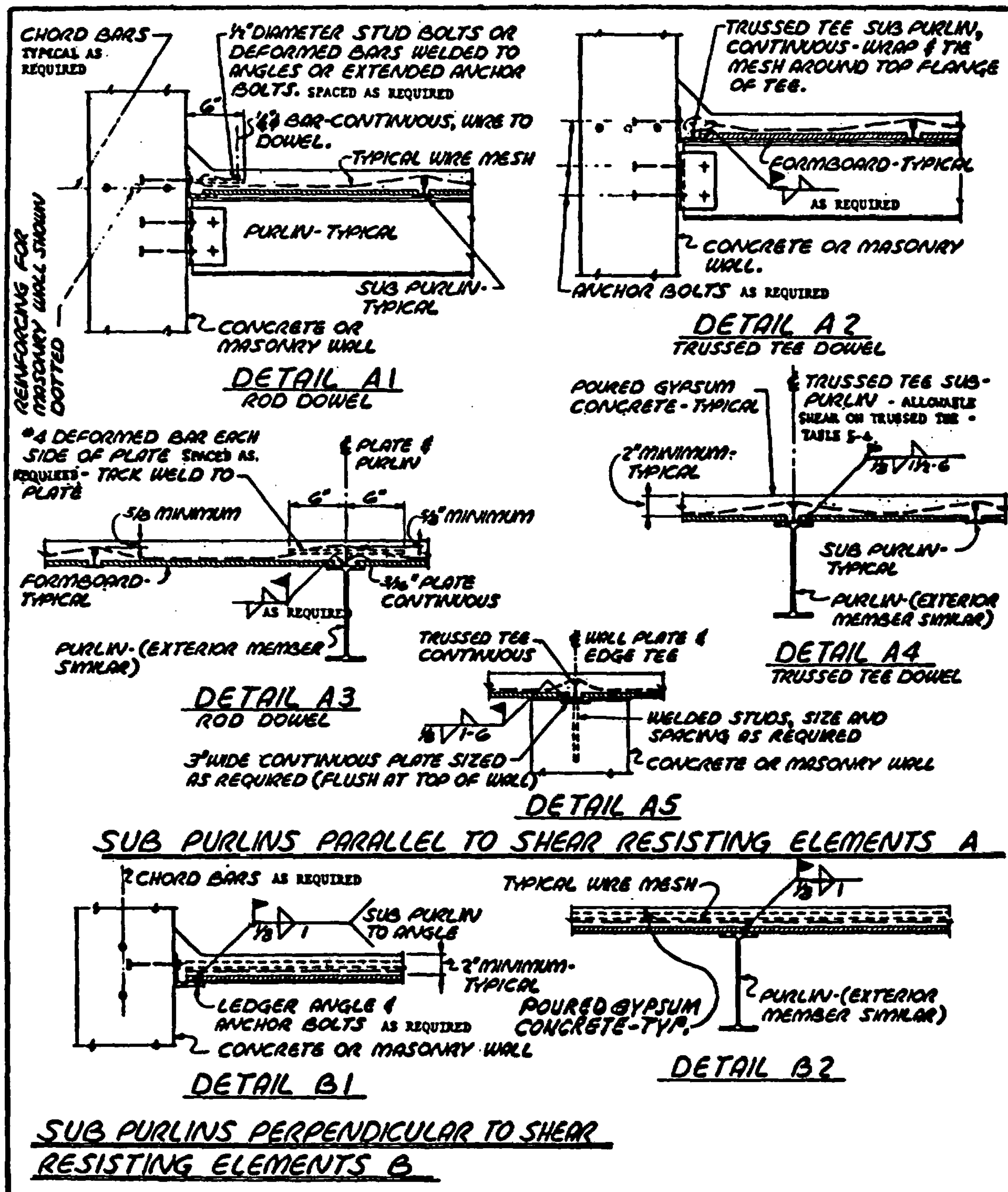
Bolt or Dowel Size (Inches)	Embedment (Inches)	Shears (Pounds)
3/8 Bolt	5	250
1/2 Bolt	5	350
5/8 Bolt	5	500
3/8 Deformed Dowel	6	250
1/2 Deformed Dowel	6	350

NOTES: *1/3 increase usually permitted on working stresses in seismic design is not applicable.
See Details A1 and A3 in Figure 5-12.

*Table 5-4. Maximum Shear on Trussed Tees**

Class A	840 pounds per foot
Class B	1,140 pounds per foot

NOTES: *1/3 increase usually permitted on working stresses in seismic design is not applicable.
See Details A2, A4, and A5 in Figure 5-12.



NOTE:

DETAILS WITH SLOPE OF ROOF SIMILAR.

Figure 5-12. Poured Gypsum Diaphragms - Typical Details

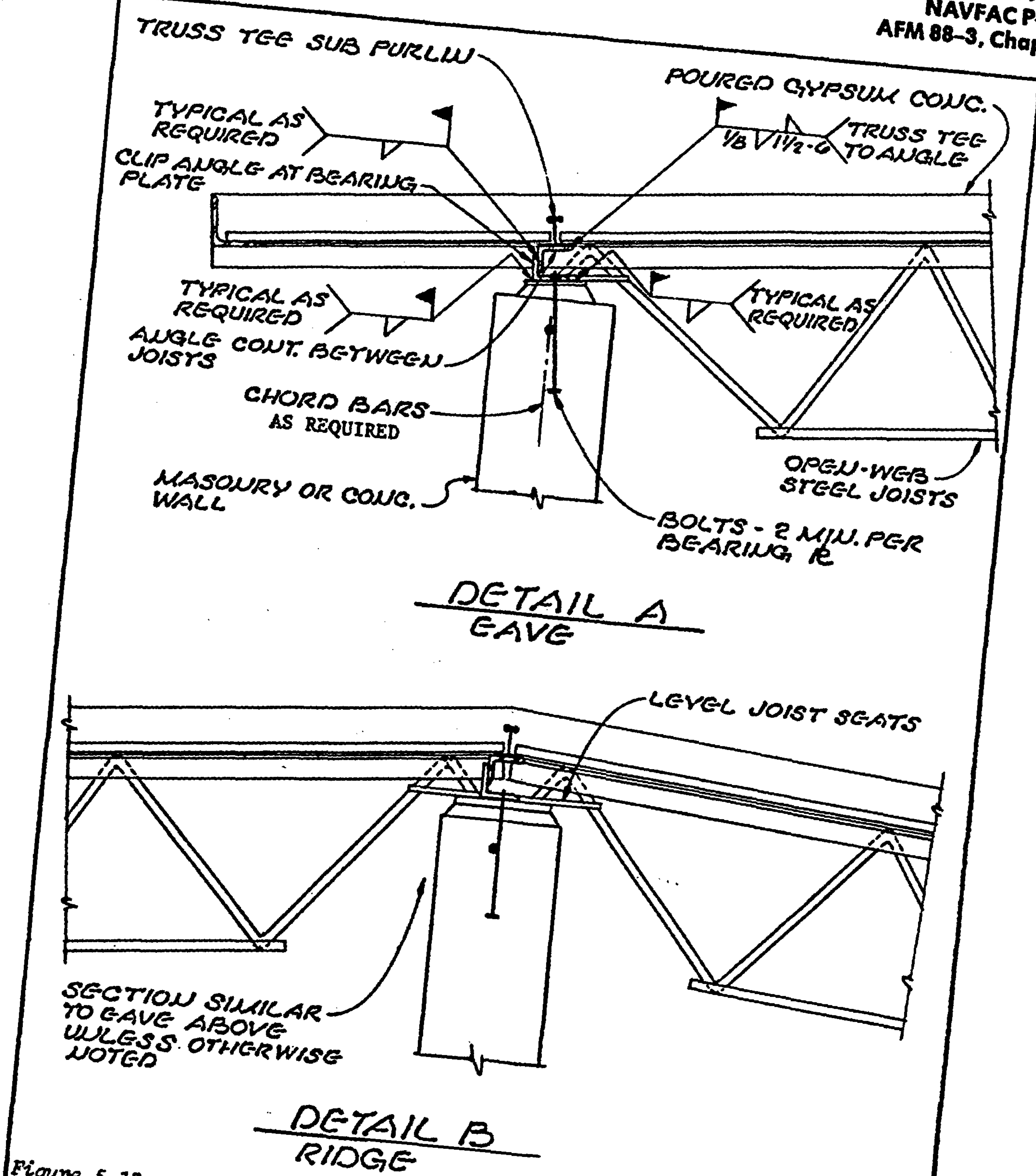


Figure 5-13. Poured Gypsum Diaphragms - Typical Details with Open-Web Joists

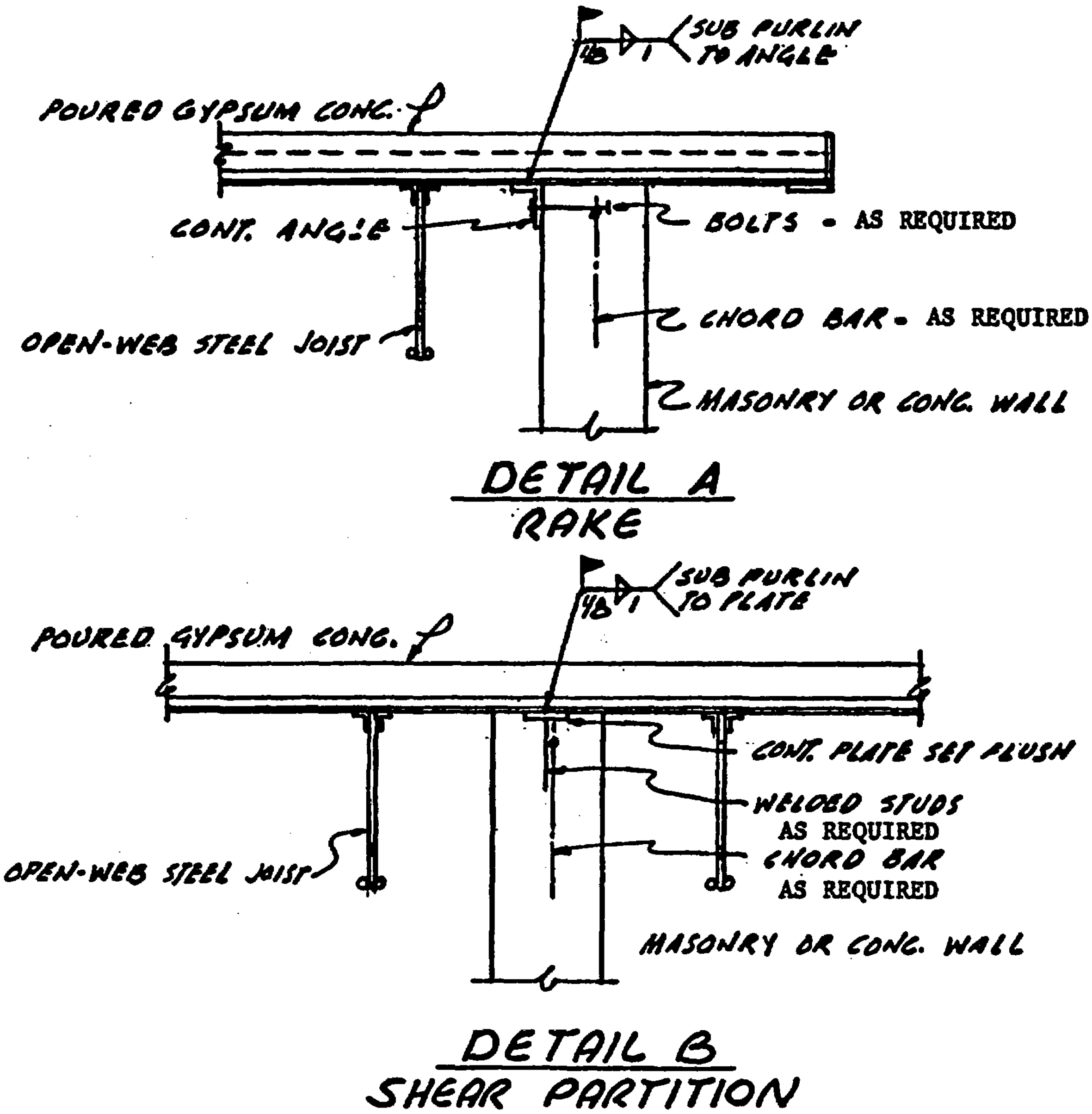


Figure 5-14. Poured Gypsum Diaphragms - Typical Details with Open-Web Joists

This indicates that the diaphragm will be in the semi-rigid category, however the span-depth and span limitations of the semi-flexible diaphragm should be used for this type of diaphragm.

d. Typical Details. Refer to figures 5-12, 5-13, and 5-14.

5-6. Steel Deck Diaphragms (Single and Multiple Sheet Decks). *a. General Design Criteria.* The following criteria will be used to design steel deck diaphragms. Three general categories of steel deck diaphragms are Type A (para 5-6*b*), Type B (para 5-6*c*), and Decks with Concrete Fill (para 5-6*d*).

(1) *Typical deck units and fastenings.* The deck units will be composed of a single fluted sheet or a combination of two or more sheets fastened together with resistance welds. The special attachments used for field attachments of steel decks are shown in figure 5-15. In addition to those shown, standard fillet (1/8 inch × 1 inch) and butt welds are also used. The depth of deck units shall not be less than 1-1/2 inches.

(2) *Definitions of special symbols.* Definitions of the special symbols used in the determination of the working shears and flexibility of steel deck diaphragms are as follows:

- a = Number of seam attachments in span L_v along a seam.
- a_p = Average spacing of profile channel closures, in feet.
- a_s = Center to center spacing of seam welds in feet. Usually L_v/a.
- a_w = Spacing of marginal welds in feet.
- b = Width of deck unit in feet.
- C_1 = 1
- C_2 = 1 for button-punched seams; $40t_s^{1/2}l'_w$ for welded seams.
- C_3 = 1 for button-punched seams; $150t_sl'_w$ for welded seams.
- C_4 = 1 for button-punched seams; $\frac{6}{L_v}$ for welded seams.
- C_5 = 1.2 for continuous angle closure; 1 for continuous zee closure; $\frac{1.44}{a_p}$ for profile channel closure.
- d = Distance in feet between outermost puddle welds attaching a deck unit to the supporting framing member.
- $F_1, F_2 \ldots$ = Components contributing to the flexibility factor F ($F = \Sigma F_n$). See paragraph 5-2*f*.
- f'_c = Compressive strength of fill concrete at 28 days in pounds per square inch.
- h = Height of fluted elements in inches (1-1/2 inch minimum).
- I_D = Gross moment of inertia of deck unit about vertical centerline axis through unit in inches to the fourth power.
- I_X = Gross moment of inertia of deck unit about the horizontal neutral axis of the deck cross-section per foot of width in inches to the fourth power.
- L_1 = Distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined.
- L_2 = Average length of each deck unit in feet.
- l_e = Length of edge lip on deck panel in inches (see Detail G in fig 5-15).
- L_R = Distance in feet between shear transfer elements.
- L_V = Vertical load span of deck units in feet.
- l_w = Minimum length in inches of seam weld.
- l'_w = Effective length in inches of seam weld. The ratio of $\frac{l'_w}{l_w}$ for the various types of seam welds is given in figure 5-16.
- n = Average number of vertical deck elements per foot which are laterally restrained at the bottom by puddle welds.
- q_D = Working shear in pounds per foot. The one-third increase usually permitted on working stresses is not applicable to this value.
- $q_1, q_2 \ldots$ = Components or limiting values of working shear in pounds per foot.
- q_{ave} = Average shear in diaphragm over length L_1 in pounds per foot.
- R = $\frac{L_v}{L_2}$
- S = Section modulus in feet of puddle weld group at supports. (Each weld assumed as unit area.)
- t_1 = Thickness of flat sheet elements in inches (22 gage minimum).
- t_2 = Thickness of fluted element in inches (22 gage minimum).
- t'_2 = Effective thickness of fluted elements in inches. See figure 5-16 for ratio of $\frac{t'_2}{t_2}$.
- t_c = Thickness of closure element in inches.
- t_f = Thickness of fill over top of deck in inches.
- t_s = Thickness in inches of deck sheet at seams.
- w = Unit weight of fill concrete in pounds per cubic foot.

(3) *Connections at ends and at supporting beams.* Refer to Type A and Type B details, paragraphs 5-6*b* and 5-6*c*.

(4) *Connections at marginal supports.* Marginal welds for all types of steel deck diaphragms will be spaced as follows:

$$a_w = \frac{35{,}000\,(t_1 + t'_2)\,C_1}{q} \text{ for puddle welds.} \quad (5\text{-}6)$$

$$a_w = \frac{1{,}200\,l'_w}{q} \text{ for fillet welds and seam welds.} \quad (5\text{-}7)$$

In no case will the spacing be greater than 3 feet. See figures 5-16 and 5-26.

(5) *Non-welded fasteners.* Fastening methods other than welds, such as self-drilling fasteners, may

be used provided that equivalence to the welded method can be shown by approved test data. The results of such test data will be presented by means of equations or tables for q_D and F in a manner similar to that used in paragraphs 5-6*b*, 5-6*c*, and 5-6*d*.

EXCEPTION: The option to fasten steel deck by powder actuated or pneumatically driven fasteners will be limited to Seismic Zone No. 1 and to areas with a wind velocity of less than 100 mph.

(6) *Maximum effective thicknesses and weld lengths.* Even though greater thicknesses and weld lengths may be installed, the maximum values for use in determining the working shears in each type of diaphragm will be as follows:

$t_1 = t_2 = t_s = .060$ inch
$t_c = .075$ inch
$l_w = 2$ inches

(7) *Thickness of steel.* The thickness of steel before coating with paint or galvanizing shall be in accordance with following table. The thickness of the uncoated steel shall not at any location be less than 95% of the design thickness.

Gage	*Design Thickness*	*Minimum Thickness*
22	0.0295	0.028
20	0.0358	0.034
18	0.0474	0.045
16	0.0598	0.057

b. Type A Diaphragms—Decks Having Shear Transfer Elements Directly Attached to Framing. Multiple plate steel decks with the flat element adjacent to framing members and single plate steel decks fall into this category of diaphragms when each deck unit is attached to the framing by at least 2 puddle welds as described on figure 5-15. t_1, t_2, t_s will not be less than 22 U.S. Standard gage. Seam attachments will be made at least at midspan of L_v but the spacing of attachments between supports will not exceed 3 feet on center. Typical details of Type A diaphragms and attachments are shown in figures 5-16, 5-17, and 5-18.

EXCEPTION to 22 gage limitation: 22 gage is the minimum thickness unless cross bracing is used to take lateral loads. However, an exception in Seismic Zone No. 1, for pre-engineered metal buildings with diaphragms less than 22 gage, requires that load tests be submitted for evaluation and approval.

(1) *Shear capacity.* The working shear will be limited to that determined by the following mulas:

$$q_D = (q_1 + q_2)\frac{q_3}{q_2}, \text{ where } \frac{q_3}{q_2} \leq C_1 \text{ but } q_D \text{ is not to} \qquad (5\text{-}8)$$

$$\text{exceed} \quad \frac{I_x \times 10^6}{2L_v^2} \qquad (5\text{-}9)$$

$$\text{nor} \quad \frac{10^4}{1.5\sqrt{L_v\left(F_1 + F_2 + \frac{F_3 L_2}{12}\right)}} \quad \text{(Applies only when } l_s < \tfrac{1}{2} \text{ inch, refer to Detail G in fig 5-15)} \qquad (5\text{-}10)$$

$$q_1 = \frac{92S(t_1 + t_2')K}{bL_v} \qquad (5\text{-}11)$$

Where K =

$$\frac{1{,}000}{\left[1 + S\left[\frac{1}{\frac{(t_1+t_2)t_1}{t_2^2} + 100n^{1/2}t_2^2\sqrt{\frac{43}{h}\left(\frac{t_2}{t_1+t_2}\right)^3}}\right]^2\right]^{1/2}} \qquad (5\text{-}12)$$

$$q_2 = \frac{abt_2^{1/2}C_2}{2}\left[q_1\left[\frac{500}{I_D} + \frac{1}{L_v dS(t_1 + t_2')^2}\right]\right]^{1/2} \qquad (5\text{-}13)$$

$$q_3 = \frac{3600t_s aC_3}{L_v} \qquad (5\text{-}14)$$

(2) *Flexibility factor.* The flexibility factor, F, will be determined by the following formulas:

$$F = F_1 + F_2 + F_3 \qquad ($$

Where

$$F_1 = \frac{1}{12(t_1 + t_2)} \qquad (5\text{-}16)$$

$$F_2 = \frac{bL_v^2C_4}{160}\left[\frac{500}{I_D} + \frac{1}{L_v dS(t_1 + t_2)^2}\right]\frac{q_1}{q_1 + q_2} \qquad (5\text{-}17)$$

$$F_3 = \frac{R}{L_v\left(t_1 + \frac{12.5n^2C_1^2t_2^3}{h}\right)} \qquad (5\text{-}18)$$

The flexibility of these diaphragms will vary within a wide range. Arrangements can be used which fall into the semi-rigid, semi-flexible, and flexible categories.

(3) *Sample calculations and tables.* A summary of allowable shear (q_d) and flexibility factors (F) for some of the more common cross-sections is shown in figure 5-19 and figure 5-20. Sample calculations using the formulas for these cross-sections are given in figures 5-21 through 5-25.

c. Type B Diaphragms—Decks Having an Elevated Plane of Shear Transfer. Multiple steel decks with fluted elements adjacent to framing members and single plate steel decks with fluted elements incapable of being welded to framing with at least two puddle welds per unit fall into this category o' phragm. This type of diaphragm has only w seam attachments. The units will be composed of

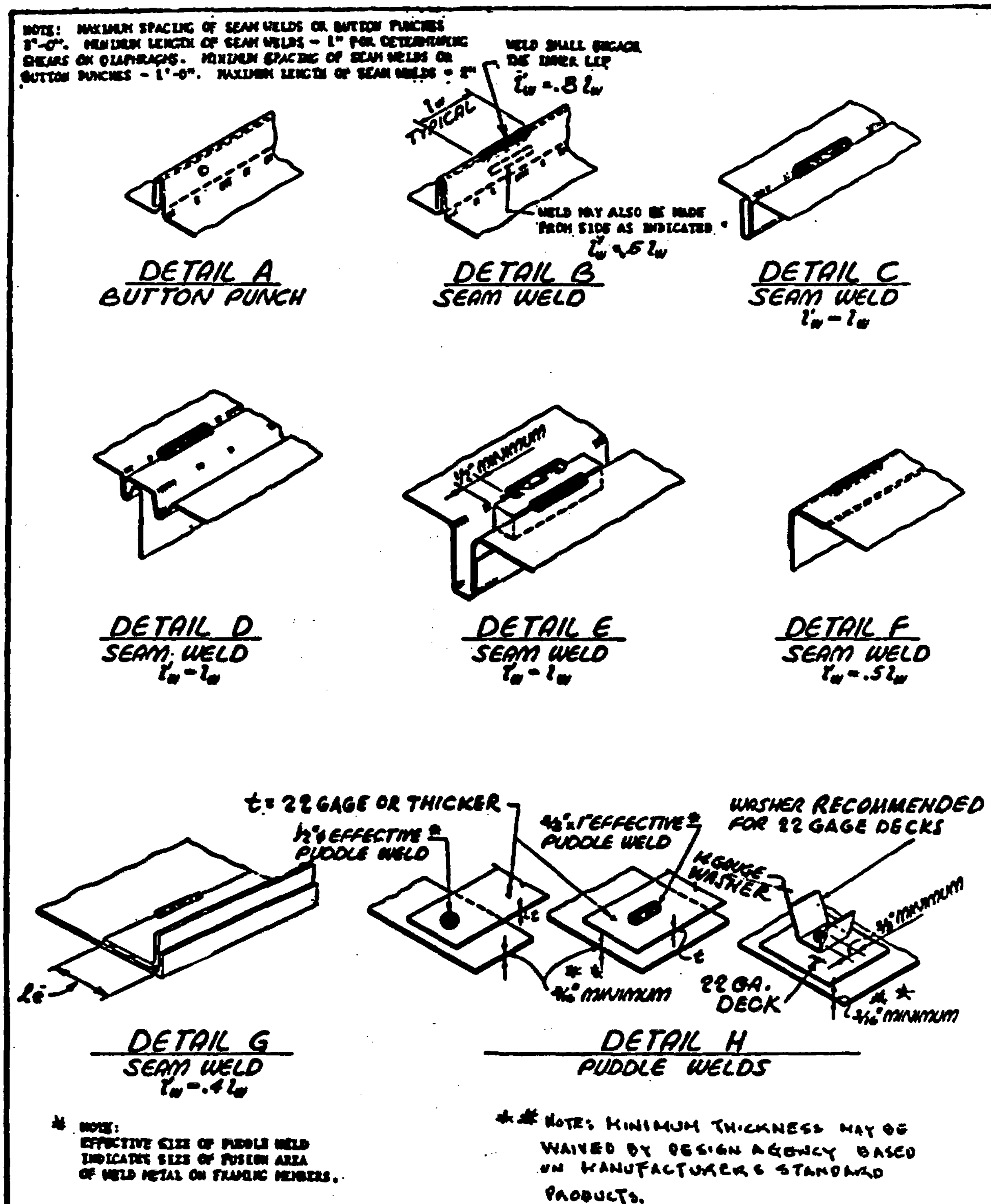
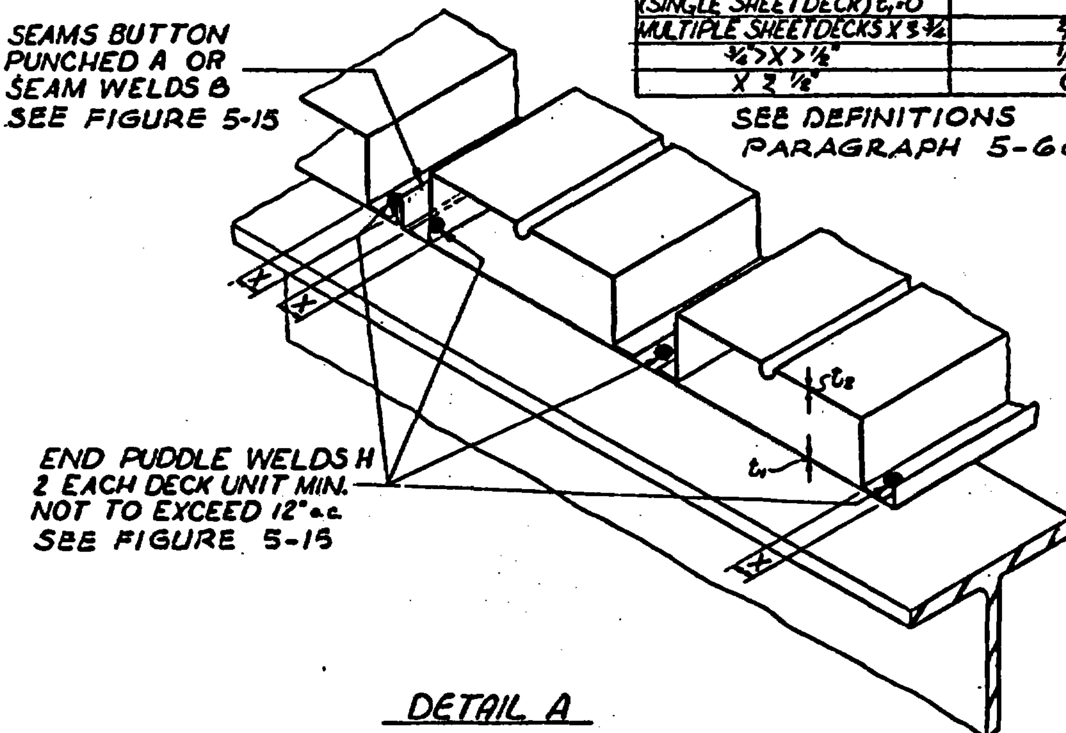


Figure 5-15. Steel Deck Diaphragms - Typical Details of Fastenings

DECK SECTION	t_2'/t_2
(SINGLE SHEET DECK) $t_1=0$	1
MULTIPLE SHEET DECKS $X \geq 3/4$	2/3
$3/4 > X > 1/2$	1/4
$X \leq 1/2$	0

SEE DEFINITIONS PARAGRAPH 5-6a (2)

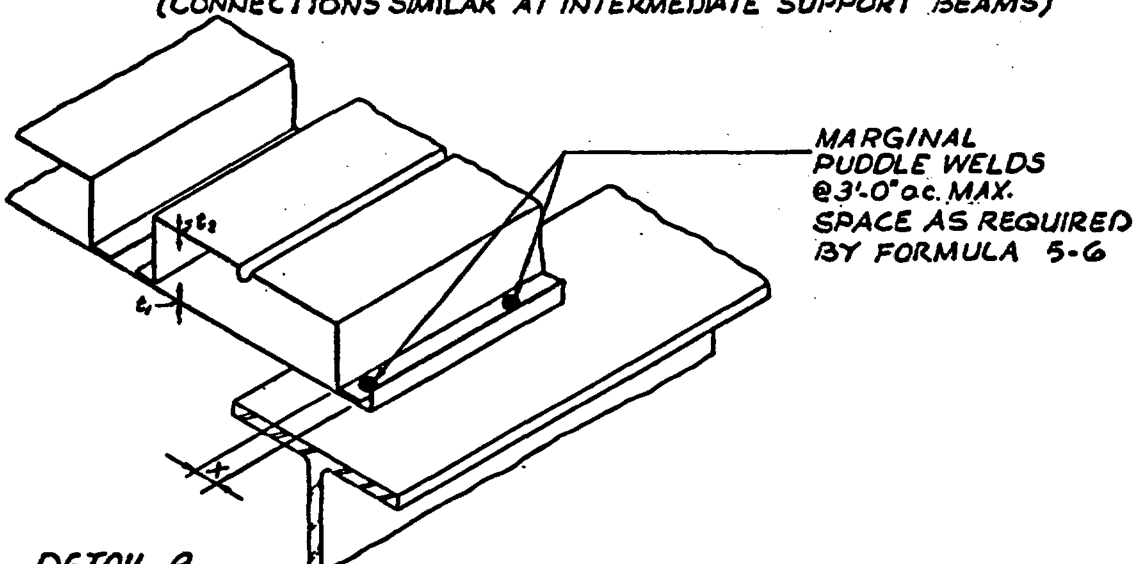


DETAIL A

END CONNECTION TO SUPPORTING BEAMS

(CONNECTIONS SIMILAR FOR DECKS WITH SINGLE SHEETS)

(CONNECTIONS SIMILAR AT INTERMEDIATE SUPPORT BEAMS)



DETAIL B

CONNECTION TO MARGINAL BEAMS

Figure 5-16. Steel Deck Diaphragms - Typical Attachments for Type A Diaphragms

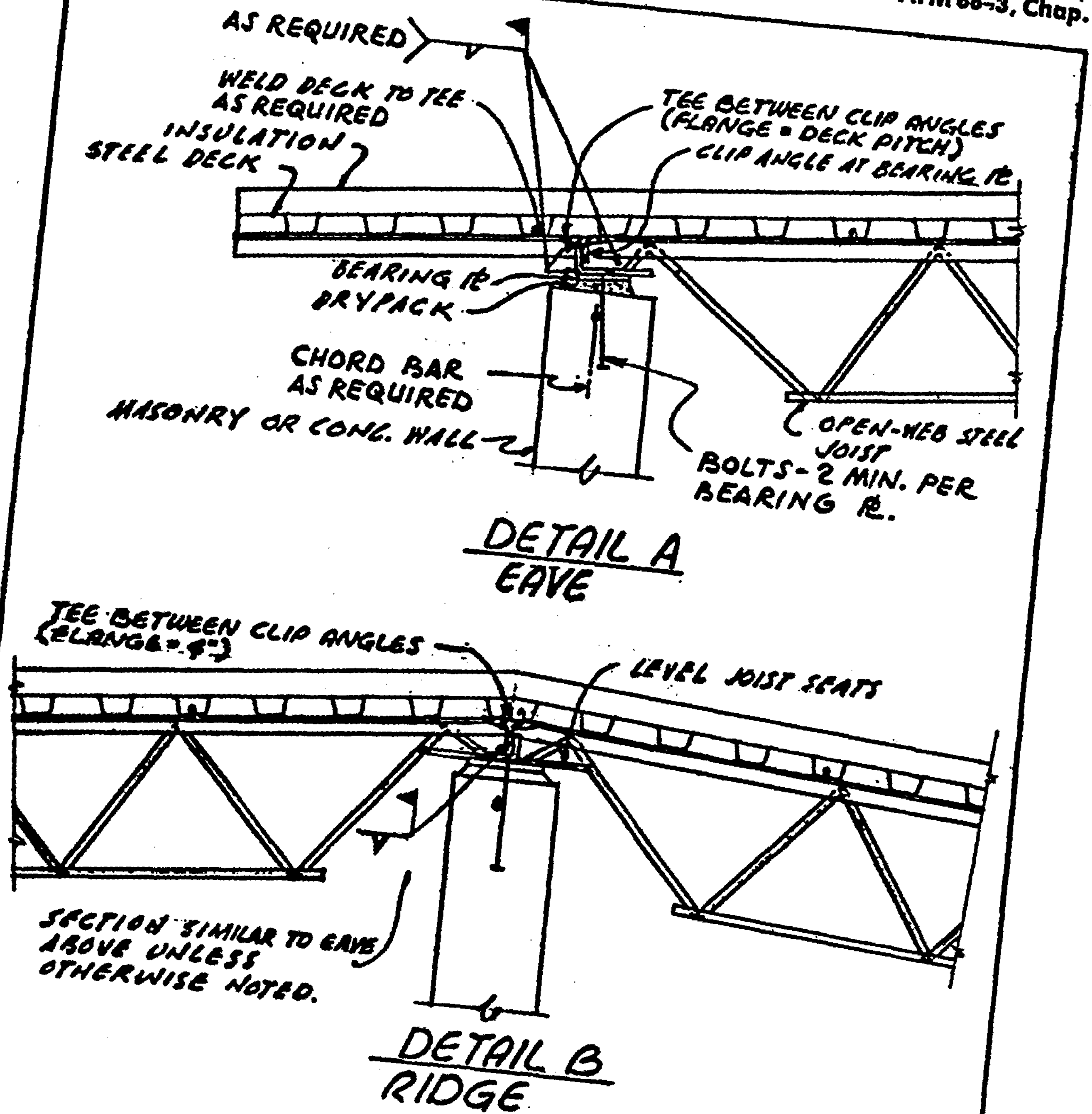


Figure 5-17. Steel Deck Diaphragms of Type A Diaphragms - Typical Details with Open-Web Joists

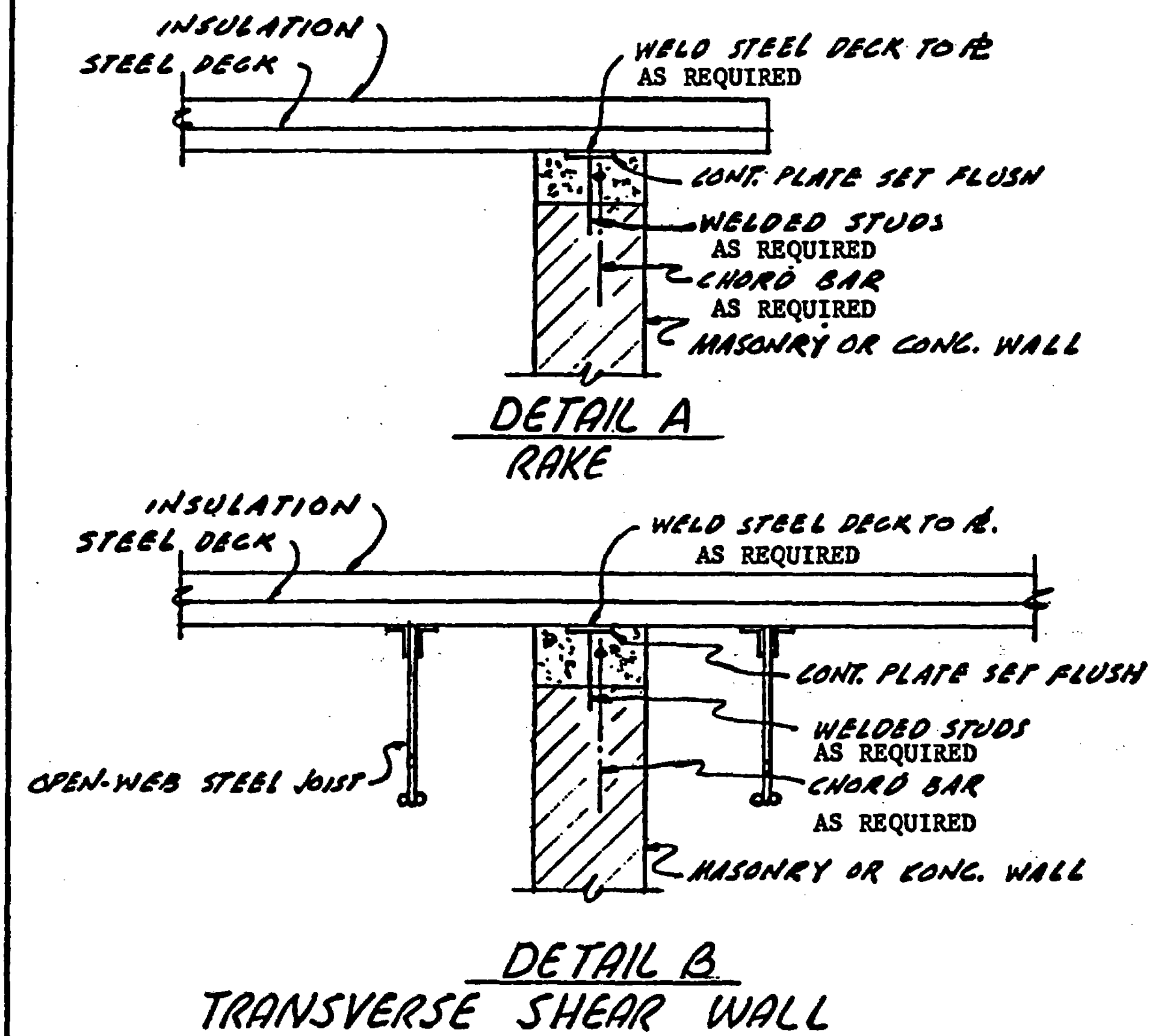
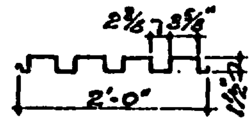
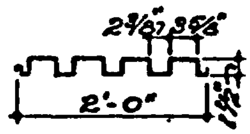
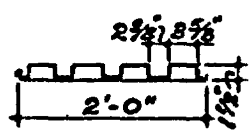
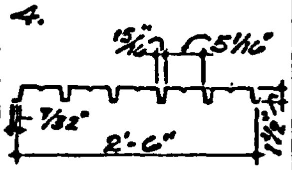


Figure 5-18. Steel Deck Diaphragm Type A - Typical Details with Open-Web Joists

TABLE OF ALLOWABLE SHEAR (q_D) AND FLEXIBILITY FACTOR (F)

SECTION	Welds *	SEAM FASTENING	GAGE		SPAN (L_v) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
1. 	3	** BUTTON PUNCH @ 24" o.c.	16	q_D	1260	1030	870	760	680	620	560
				F	5.7+ 34.7R	7.0+ 27.8R	8.3+ 23.1R	9.6+ 19.8R	11+ 17.4R	12+ 15.4R	14+ 13.9R
			18	q_D	900	740	630	550	500	450	410
				F	8.1+ 67.8R	9.9+ 54.2R	12+ 45.2R	13+ 38.7R	15+ 33.9R	17+ 30.1R	19+ 27.1R
			20	q_D	520	430	370	320	290	260	240
				F	13+ 161R	15+ 129R	18+ 107R	21+ 91.9R	25+ 60.4R	26+ 71.4R	28+ 64.3R
			22	q_D	340	280	240	210	190	180	160
				F	17+ 278R	20+ 222R	23+ 185R	27+ 159R	30+ 139R	32+ 123R	35+ 111R
2. 	5	** BUTTON PUNCH @ 24" o.c.	16	q_D	1650	1340	1130	980	870	790	720
				F	5.0+ 8.65R	6.1+ 6.94R	7.3+ 5.79R	8.5+ 4.96R	9.8+ 4.34R	11+ 3.86R	13+ 3.47R
			18	q_D	1220	990	840	730	660	580	520
				F	7.1+ 17.0R	8.6+ 13.6R	10+ 11.3R	12+ 9.69R	14+ 8.48R	15+ 7.54R	17+ 6.78R
			20	q_D	700	560	470	410	360	320	290
				F	11+ 40.2R	13+ 32.1R	16+ 26.8R	18+ 23.0R	21+ 20.1R	23+ 17.9R	26+ 16.1R
			22	q_D	450	370	310	270	240	220	200
				F	15+ 69.4R	18+ 55.5R	21+ 46.3R	24+ 39.7R	27+ 34.7R	30+ 30.9R	32+ 27.8R
3. 	3	** BUTTON PUNCH @ 24" o.c.	18-18	q_D	1580	1280	1080	930	830	750	680
				F	3.5+ 3.98R	4.4+ 3.18R	5.3+ 2.65R	6.2+ 2.28R	7.2+ 1.99R	8.3+ 1.77R	9.4+ 1.59R
			16-16	q_D	1990	1610	1360	1170	1040	930	850
				F	2.5+ 2.81R	3.1+ 2.25R	3.8+ 1.87R	4.5+ 1.61R	5.2+ 1.41R	6.0+ 1.25R	6.9+ 1.13R
			16-18	q_D	1920	1550	1310	1130	1000	900	820
				F	2.9+ 3.34R	3.6+ 2.67R	4.4+ 2.22R	5.2+ 1.91R	6.1+ 1.67R	7.0+ 1.49R	8.0+ 1.34R
			20-20	q_D	1180	960	810	690	600	530	480
				F	5.6+ 5.92R	6.9+ 4.73R	8.2+ 3.94R	9.7+ 3.38R	11+ 2.96R	13+ 2.63R	14+ 2.37R
4. 	6	1½" SEAM WELD @ 18" o.c.	18	q_D	990	890	820	760	710	680	650
				F	6.7+ 17.0R	5.5+ 13.6R	5.4+ 11.3R	5.3+ 9.68R	5.2+ 8.47R	5.1+ 7.53R	5.0+ 6.78R
			20	q_D	710	640	590	550	520	490	460
				F	8.5+ 40.2R	8.1+ 32.1R	7.8+ 26.8R	7.6+ 23.0R	7.3+ 20.1R	7.1+ 17.9R	6.9+ 16.1R
			22	q_D	480	420	380	350	330	310	300
				F	11+ 69.4R	10+ 55.5R	9.7+ 46.3R	9.3+ 39.7R	8.9+ 34.7R	8.6+ 30.9R	8.3+ 27.8R

5. See Figure 5-20

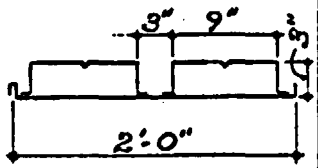
** SEAM WELDS ARE PREFERABLE.

*Number of welds at end and at intermediate support beams.

NOTE:
THE GAGES FOR MULTIPLE SHEET DECKS ARE DESIGNATED WITH THE GAGE OF THE FLAT SHEET FIRST AND FLUTED SHEET SECOND.

Figure 5-19. Steel Deck Diaphragm Type A - Allowable Shears and Flexibility Factors

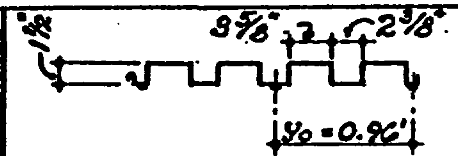
TABLE OF ALLOWABLE SHEAR (q_D) AND FLEXIBILITY FACTOR (F)

SECTION	END WELDS	SEAM FASTENING	GAGE		SPAN (Lv) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"
5. 	3	BUTTON PUNCH @ 24" o.c.	18-18	q_D	1180	960	810	710	630	570	520	480	450	420
				F	5.1+ 5.01R	6.2+ 4.01R	7.4+ 3.34R	8.7+ 2.87R	9.9+ 2.51R	11+ 2.23R	13+ 2.01R	14+ 1.82R	15+ 1.67R	17+ 1.54R
			16-16	q_D	1480	1200	1010	880	780	700	640	590	550	520
				F	3.5+ 3.93R	4.3+ 3.14R	5.2+ 2.62R	6.1+ 2.25R	7.0+ 1.97R	8.0+ 1.75R	9.0+ 1.57R	.10+ 1.43R	.11+ 1.31R	.12+ 1.21R
			18-16	q_D	1160	940	800	690	620	560	510	470	440	410
				F	4.5+ 4.84R	5.6+ 3.87R	6.7+ 3.22R	7.8+ 2.77R	8.9+ 2.42R	.10+ 2.15R	.11+ 1.94R	12+ 1.76R	14+ 1.61R	.15+ 1.49R
			16-18	q_D	1510	1230	1040	900	800	720	660	610	570	530
				F	3.8+ 4.04R	4.7+ 3.23R	5.7+ 2.69R	6.7+ 2.31R	7.7+ 2.02R	8.8+ 1.80R	9.9+ 1.62R	11+ 1.47R	12+ 1.35R	13+ 1.24R

NOTE:
THE GAGES FOR MULTIPLE SHEET DECKS ARE DESIGNATED WITH THE GAGE OF THE FLAT SHEET FIRST AND FLUTED SHEET SECOND.

Figure 5-20. Steel Deck Diaphragm Type A - Allowable Shears and Flexibility Factors

SAMPLE CALCS. NO. 1 FOR TYPE A DIAPHRAGM.



20 GAGE SINGLE PLATE DECK, BUTTON PUNCH SEAMS @ 24" o.c., 3 END WELDS

$t_1 = 0$

$t_2 = t_2' = t_s = 0.036"$

$S = \frac{\Sigma y^2}{y_o} = \frac{2 \times .96^2}{.96} = 1.92'$

$I_x = .23$

$b = 2'$ $\quad h = 1.5"$

$L_v = 10'\text{-}0"$ $\quad a = L_v/2$

$n = 4/2 = 2$ $\quad d = 1.92 = 2y_o$

$I_D = 68$

$C_1 = C_2 = C_3 = C_4 = 1$

$$q_o = (q_1 + q_2)\frac{q_3}{q_2} \qquad \text{(PARA. 5-6b)}$$

$$q_1 = \frac{92\,S(t_1 + t_2')K}{b L_v}$$

$$q_2 = \frac{a b t_s^{1/2} C_2}{2}\left\{q_1\left[\frac{500}{I_D} + \frac{1}{L_v d S (t_1 + t_2')^2}\right]\right\}^{1/2}$$

$$K = \frac{1000}{\left\{1 + S\left[\frac{1}{\frac{(t_1+t_2)t_1}{t_2^2} + 100\,n^{1/2} t_2^2 \sqrt{\frac{t_2}{h}}\left(\frac{t_2}{t_1+t_2}\right)^3}\right]^2\right\}^{1/2}}$$

$$q_3 = \frac{3600\,t_s\,a\,C_3}{L_v}$$

$$K = \frac{1000}{\left\{1 + 1.92\left[\frac{1}{100\sqrt{2}\,(.036)^2\sqrt{\frac{.036}{1.5}}}\right]^2\right\}^{1/2}} = \frac{1000}{1.73} = 578$$

$$q_1 = \frac{92 \times 1.92 \times .036 \times 578}{2 \times 10} = 184$$

$$q_2 = \frac{5 \times 2\sqrt{.036}}{2}\left\{184\left[\frac{500}{68} + \frac{1}{10 \times 1.92 \times 1.92 \times (.036)^2}\right]\right\}^{1/2}$$

$$= .945\sqrt{5230} = 68.4$$

$$q_3 = \frac{3600 \times .036 \times 5}{10} = 64.8 \qquad \frac{q_3}{q_2} = \frac{64.8}{68.4} = 0.95$$

$$q_o = (184 + 68.4).95 = 240$$

$$\frac{I_x \times 10^6}{2L_v^2} = \frac{.23 \times 10^6}{2 \times 10^2} = 1150 > 240 \ \underline{\text{O.K.}}$$

$q_o = 240$ (FIGURE 5-19: $L_v = 10'$, 20 ga.)

$$F = F_1 + F_2 + F_3$$

$$F_1 = \frac{1}{12(t_1 + t_2)} = \frac{1}{12 \times .036} = 2.32$$

$$F_2 = \frac{b L_v^2 C_4}{160}\left[\frac{500}{I_D} + \frac{1}{L_v d S (t_1 + t_2')^2}\right]\frac{q_1}{q_1 + q_2} = \frac{2 \times 100}{160}\left[7.35 + 21.1\right]\frac{184}{252.4} = 25.9$$

$$F_3 = \frac{R}{L_v\left(t_1 + \frac{12.5 n^2 t_2^3 C_1^2}{h}\right)} = \frac{R}{10\left(\frac{12.5 \times 4 \times .036^3}{1.5}\right)}$$

$$= \frac{R}{10 \times .00156} = 64R$$

$$F = 2.32 + 25.9 + 64R = 28.2 + 64R$$

(SEE FIGURE 5-19: $L_v = 10'$, 20 ga.)

Figure 5-21. Steel Deck Diaphragm Type A - Sample Calculation No. 1

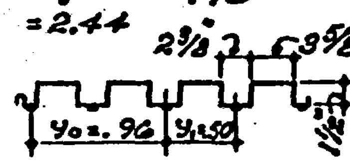
SAMPLE CALC. NO. 2 FOR TYPE A DIAPHRAGM

18 GAGE SINGLE PLATE DECK.
BUTTON PUNCH SEAMS @ 24" o.c.
5 END WELDS
L_v - 9'-0"

$t_1 = 0$

$t_2 = t_2' = t_3 = .048"$

$S = \frac{\Sigma y^2}{y_0} = \frac{2 \times .96^2 + 2 \times .50^2}{.96} = 2.44$



$b = 2'$ $\quad h = 1.5"$

$L_v = 9'$ $\quad I_x = .34$

$n = 8/2 = 4$ $\quad a = L_v/2$

$I_0 = 91$ $\quad d = 1.92 = 240$

$C_1 = C_2 = C_3 = C_4 = 1$

$$K = \frac{1000}{\left\{1 + 2.44\left[\frac{1}{100 \times 2(.048)^2\sqrt{\frac{43}{1.5}}}\right]^2\right\}^{1/2}} = \frac{1000}{1.18} = 845$$

$$q_1 = \frac{92 \times 2.44 \times .048 \times 845}{2 \times 9} = 505$$

$$q_2 = \frac{9 \times .048}{2}^{1/2}\left\{505\left[\frac{500}{91} + \frac{1}{9 \times 1.92 \times 2.44(.048)^2}\right]\right\}^{1/2} = 87.9$$

$$q_3 = \frac{3600 \times .048 \times 4.5}{9} = 86.5 \qquad \frac{q_3}{q_2} = \frac{86.5}{87.9} = .985$$

$$q_D = (505 + 87.9).985 = \underline{584}$$

$$\frac{I_x \times 10^6}{2L_v^2} = \frac{.34 \times 10^6}{2 \times 9^2} = 2099 > 584 \ \underline{\text{O.K.}}$$

$q_D = 580$ (FIGURE 5-19: $L_v = 9'$, 18 ga.)

$$F_1 = \frac{1}{12 \times .048} = 1.74$$

$$F_2 = \frac{2 \times 9^2}{160}(15.8)\frac{505}{592.9} = 13.7$$

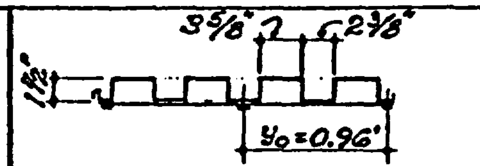
$$F_3 = \frac{R}{9\left(\frac{12.5 \times 16(.048)^3}{1.5}\right)} = \frac{R}{9 \times .0147} = 7.55R$$

$$F = 1.74 + 13.7 + 7.55R = \underline{15.4 + 7.6R}$$

(SEE FIGURE 5-19: $L_v = 9'$, 18 ga.)

Figure 5-22. Steel Deck Diaphragm Type A - Sample Calculation No. 2

SAMPLE CALCS. NO. 3 FOR TYPE A DIAPHRAGM



16-18 GAGE MULTIPLE PLATE DECK, BUTTON PUNCH SEAMS @ 24" o.c., 3 END WELDS.

$t_1 = 0.048 \quad t_2' = 2/3 \times .048 = .032$

$S = \frac{\Sigma y^2}{y_o} = \frac{2 \times .96^2}{.96} = 1.92$

$I_x = .603$

$b = 2' \qquad h = 1.5$

$L_v = 8'\text{-}0" \qquad a = L_v/2$

$n = \frac{4}{2.0'} = 2 \qquad d = 1.92 = 2y_o$

$I_D = 155$

$C_1 = C_2 = C_3 = C_4 = 1$

$$q_3 = \frac{3600 \times .060 \times 4}{8} = 108$$

$$\frac{q_3}{q_2} = \frac{108}{79.6} = > 1$$

(NOTE: q_2 COMPUTED BELOW)

$$K = \frac{1000}{\left\{1 + 7.92\left[\frac{1}{\frac{(.060+.048)(.060)}{(.048)^2} + 100 \times 2 \times (.048)^2 \sqrt{\frac{45}{1.5}}\left(\frac{.048}{.048+.06}\right)^3}\right]^2\right\}^{1/2}}$$

$$= \frac{1000}{1.1} = 910$$

$$q_1 = \frac{92 \times 1.92 \times .092 \times 910}{2 \times 8} = 920$$

$$q_2 = \frac{8 \times .060^{1/2}}{2}\left\{920\left[\frac{500}{155} + \frac{1}{8 \times 1.92 \times 1.92\,(.092)^2}\right]\right\}^{1/2} = 79.6$$

$$q_o = 920 + 79.6 = 999.6$$

$$\frac{I_x \times 10^6}{2 L_v^2} = \frac{.603 \times 10^6}{2 \times 8^2} = 4711 > 999.6 \ \underline{\text{O.K.}}$$

$q_o = 1000$ (FIGURE 5-19: $L_v = 8'$, 16-18 ga.)

$$F_1 = \frac{1}{12 \times .108} = 0.77$$

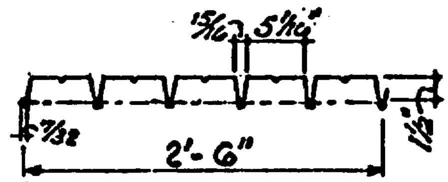
$$F_2 = \frac{2 \times 64}{160}(7.23)\frac{920}{999.6} = 5.33$$

$$F_3 = \frac{R}{8\left[.060 + \left(\frac{12.5 \times 16 \times .048^2}{1.5}\right)\right]} = \frac{R}{8 \times .0747} = 1.67R$$

$F = 0.77 + 5.33 + 1.67R = 6.1 + 1.7R$
(FIGURE 5-19: $L_v = 8'$, 16-18 ga.)

Figure 5-23. Steel Deck Diaphragm Type A - Sample Calculation No. 3

SAMPLE CALCS. NO. 4 FOR TYPE A DIAPHRAGM



18 GAGE SINGLE PLATE DECK, 1 1/2" WELDED SEAMS @ 18" o.c.
6 END WELDS

$t_1 = 0$

$t_2 = t_2' = t_3 = .048$

$$S = \frac{\Sigma y^2}{y_0} = \frac{2 \times .25^2 + 2 \times .75^2 + 2 \times 1.25^2}{1.25} = 3.5$$

$I_x = .212$

$b = 2.5$ $\qquad h = 1.5$

$L_v = 5'$ $\qquad a = 5/1.5 = 3.33$

$n = \frac{10}{2.5} = 4$ $\qquad d = 2.5 = 2y_0$

$I_D = 189$ $\qquad l_w' = .4 l_w = .4 \times 1.5 = .6$

$C_1 = 1$

$C_2 = 40 t_3^{1/2} l_w' = 40 \times .048^{1/2} \times .6 = 5.26$

$C_3 = 150 t_3 l_w' = 150 \times .048 \times .6 = 4.32$

$C_4 = 6/L_v = 6/5 = 1.2$

$$K = \frac{1000}{\left\{1 + 3.5\left[\frac{1}{100 \times 2(.048)^2\sqrt{\frac{4.3}{1.5}}}\right]^2\right\}^{1/2}} = 796$$

$$q_1 = \frac{92 \times 3.5 \times .048 \times 796.0}{2.5 \times 5} = 984.2$$

$$q_2 = \frac{3.33 \times 2.5 \times .048^{1/2} \times 5.26}{2}\left\{984.2\left[\frac{500}{189} + \frac{1}{5 \times 2.5 \times 3.5 \times .048^2}\right]\right\}^{1/2} = 535$$

$$q_3 = \frac{3600 \times .048 \times 3.33 \times 4.32}{5} = 497.2$$

$$\frac{q_3}{q_2} = \frac{497.2}{535} = .93$$

$$q_D = (984.2 + 535).93 = 1413$$

$$\frac{I_x \times 10^6}{2L_v^2} = \frac{.212 \times 10^6}{2 \times 5^2} = 4240 > 1413 \text{ O.K.}$$

$$F_1 = \frac{1}{12 \times .048} = 1.74$$

$$F_2 = \frac{2.5 \times 5^2 \times 1.2}{160}\left[\frac{500}{189} + \frac{1}{5 \times 2.5 \times 3.5 \times .048^2}\right]\frac{984.2}{984.2 + 535} = 3.84$$

$$F_3 = \frac{R}{5\left(\frac{12.5 \times 16 \times .048^3}{1.5}\right)} = 13.6R$$

$$F = 1.74 + 3.84 + 13.6R = 5.58 + 13.6R$$

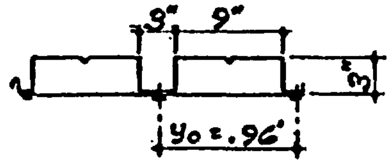
(FIGURE 5-19 : $L_v = 5'$, 18 ga.)

$$q_D = \frac{10^4}{1.5\sqrt{5\left(5.58 + \frac{13.6 \times 5}{12}\right)}} = 889.0 \text{ (EQUATION 5-6-5)} < 1413$$

q_D = 890 (FIGURE 5-19 : L_v = ~~5', 18 ga.~~)

Figure 5-24. Steel Deck Diaphragm Type A - Sample Calculation No. 4

SAMPLE CALCS. NO.5 FOR TYPE A DIAPHRAGM



16-18 GAGE MULTIPLE PLATE DECK, BUTTON PUNCH SEAMS @ 24" o.c. 3 END WELDS.

$t_1 = t_3 = .060$

$t_2 = .048 \quad t_2' = 1/4 \times .048 = .012$

$S = \frac{\Sigma y^2}{y_o} = \frac{2 \times .96^2}{.96} = 1.92$

$I_x = 2.35$

$b = 2' \qquad h = 3$

$L_v = 10' \qquad a = L_v/2$

$n = \frac{4}{2.0} = 2 \qquad d = 1.92 = 2y_o$

$I_D = 160$

$C_1 = C_2 = C_3 = C_4 = 1$

$$q_s = \frac{3600 \times .060 \times 5}{10} = 108$$

$$\frac{q_s}{q_2} = \frac{108}{84.6} = > 1.0$$

(NOTE: q_2 COMPUTED BELOW)

$$K = \frac{1000}{\left\{1 + 1.92\left[\frac{1}{2.81 + (.326 \times 3.78 \times .0882)}\right]^2\right\}^{1/2}} = \frac{1000}{\sqrt{1 + .226}} = \frac{1000}{1.107} = 904$$

$$q_1 = \frac{92 \times 1.92 (.072)\, 904}{2 \times 10} = 575$$

$$q_2 = \frac{10 \times .060^{1/2}}{2}\left[575\left(\frac{500}{160} + \frac{1}{10 \times 1.92 \times 1.92 (.072)^2}\right)\right]^{1/2} = 84.6$$

$$q_o = (575 + 84.6)\, 1.0 = 659.6$$

$$\frac{I_x \times 10^6}{2 L_v^2} = \frac{2.35 \times 10^6}{2 \times 10^2} = 11750 > 659.6 \;\; \underline{\text{O.K.}}$$

$q_o = 660$ (FIGURE 5-20: $L_v = 10'$, 16-18 ga.)

$$F_1 = \frac{1}{12 \times .108} = 0.77$$

$$F_2 = \frac{2 \times 10^2}{160}(8.36) \times \frac{575}{660} = 9.70$$

$$F_3 = \frac{R}{10\left(.060 + \frac{12.5 \times 4 (.048)^3}{3}\right)} = \frac{R}{10 \times .0618} = 1.62R$$

$$F = .77 + 9.10 + 1.62R = 9.87 + 1.62R$$

(FIGURE 5-20: $L_v = 10'$, 16-18 ga.)

Figure 5-25. Steel Deck Diaphragm Type A - Sample Calculation No. 5

sheets not less than 20 U.S. Standard gage. Seam attachment spacing will not exceed 3 feet on center. Typical details of Type B diaphragms and attachments are shown in figures 5-26 through 5-28.

(1) *Shear capacity.* The working shear will be limited to that determined by the following formulas:

$$q_D = q_3, q_4, \text{ or } q_5, \text{ whichever is the lesser, but not to exceed 1,050 pounds per foot.} \quad (5\text{-}19)$$

$$q_3 = \frac{0.6 t_s^2 a l'_w}{L_v} \quad (5\text{-}20)$$

$$q_4 = \frac{t_s}{10}\left(\frac{1}{a_s}\right)^2 \times 10^6 \quad (5\text{-}21)$$

$$q_5 = \frac{C_5 t_c^2 \times 10^6}{2h^{1/4}} \quad (5\text{-}22)$$

(2) *Flexibility factor.* The flexibility factor, F, will be determined by the following formulas:

$$F = F_1 + F_4 + F_5 \quad (5\text{-}23)$$

Where

$$F_1 = \frac{1}{12(t_1+t_2)} \quad (5\text{-}24)$$

$$F_4 = \frac{3{,}500}{q_3} \quad (5\text{-}25)$$

$$F_5 = \frac{20{,}000}{L_R q_5} \quad (5\text{-}26)$$

The flexibility of these diaphragms will fall into the semi-rigid and semi-flexible categories.

d. Steel Decks with Concrete Fill. This type of diaphragm is composed of a galvanized steel deck with a superimposed fill of concrete having a minimum f'_c of 2,500 p.s.i. at 28 days and a minimum w of 90 pounds per cubic foot. Minimum concrete fill over the deck will be 2-1/2 inches. Temperature reinforcement will be used in the fill with the minimum area of 6×6/#10-#10. Steel decks less than 1-1/2 inches in depth do not qualify as diaphragms, thus only the concrete is considered as the diaphragm per paragraph (1) below. To satisfy anchorage requirements required in paragraph 5-4*b*, positive interlocking between the steel deck and the concrete can be achieved by either deck embossments or indentations, transverse wires attached to the deck corrugations, holes placed in the corrugations, or deck profile in which the fluted elements are placed up so that the fill is keyed with the deck. If interlocking between the deck and the concrete is not achieved, then mechanical anchorages will be required to anchor the fill to the supporting member as prescribed in paragraph 5-4*b*(2).

(1) *Concrete as a diaphragm.* If the diaphragm is loaded and reacted without shear stresses passing through the deck or its attachments, the diaphragm is a concrete diaphragm as described in paragraph 5-4. Typical attachment details are shown in fig 5-29, Details A and B.

(2) *Steel deck as a diaphragm*

(a) Shear capacity. If the diaphragm shears pass through the deck and its attachments, the working shear will be determined by the following formulas:

$$q_D = q_1 + q_6 \quad (5\text{-}27)$$

Where

$$q_1 = \frac{92S(t_1+t'_2)K}{bL_v} \quad \text{in which } K = 1{,}000 \quad (5\text{-}28)$$

$$q_6 = q'_6 + q''_6 \quad (5\text{-}29)$$

Where

$$q'_6 = \frac{t_f w^{1.5}\sqrt{f'_c}}{200} \quad (5\text{-}30)$$

And

$$q''_6 = 2\sqrt{\frac{KB}{d(t+t'_2)}} \quad (5\text{-}31)$$

(b) Flexibility factor. The flexibility factor, F, will be determined by using the formula:

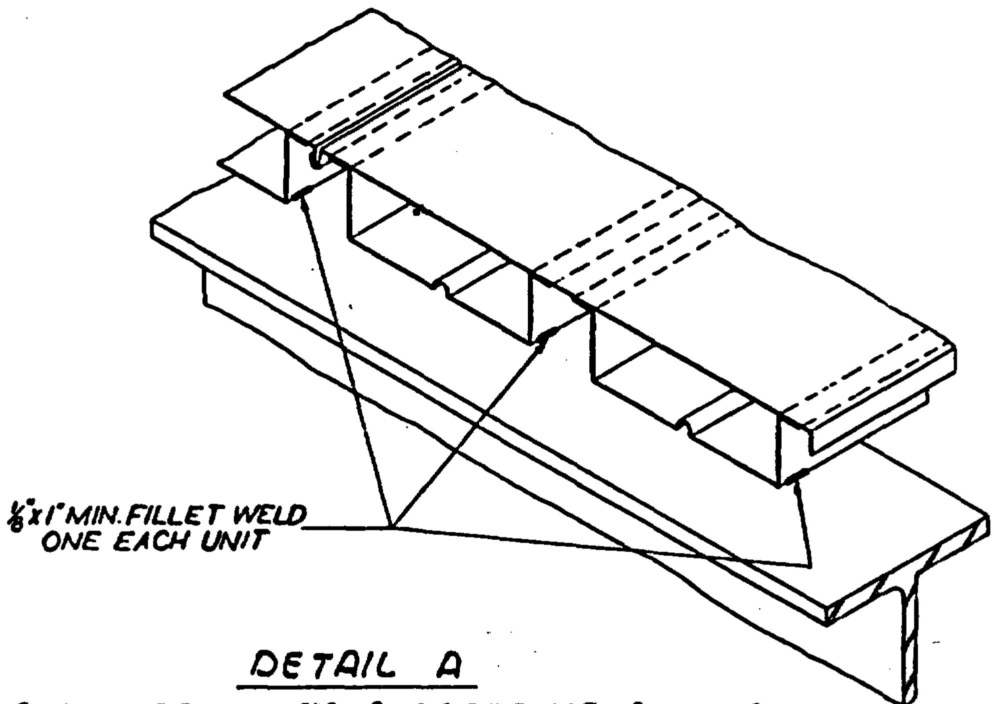
$$F = \frac{20q''_6}{b^2 q_D} \quad (5\text{-}32)$$

The flexibility of these diaphragms usually falls in the rigid category.

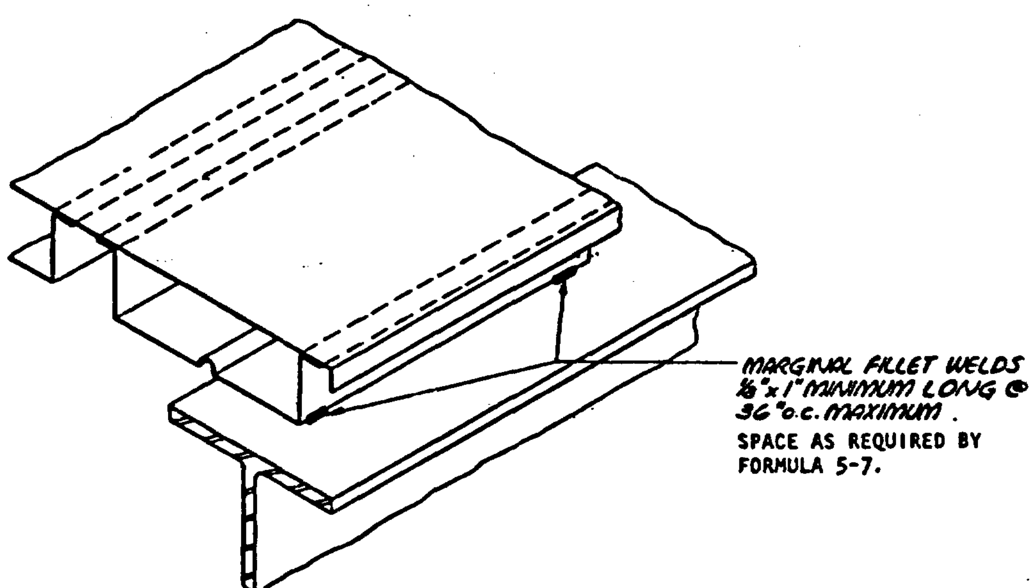
(c) Sample calculation and table. Typical attachment details are shown in figure 5-29, Details C and D. A summary of allowable shears (q_d) and flexibilities (F) for a typical cross-section is shown in figure 5-30. A solution to the formulas for a typical cross-section of this type of diaphragm is given in figure 5-31.

5-7. Wood Diaphragms. *a. General Design Criteria.* The following criteria will be used to design wood diaphragms. (Also, refer to chap 3, para 3-3(J)3b.)

(1) *Straight sheathing.* Straight sheathing diaphragms will be constructed of one- or two-inch nominal boards, six or eight inches nominal in width with boards laid at right angles to the rafters or joists. Boards will be nailed to each rafter or joist and peripheral blocking using two 8d common nails for 1-inch×6-inch and 1-inch×8-inch sheathing. For 2-inch sheathing, nails will be three 16d. End joints of adjacent boards will be separated by at least two joist or rafter spaces with at least two boards between joints on same support. The diaphragm shear value will be as indicated in table 5- Diaphragms of this category will have a value o (see para 5-2*f* and table 5-1) in the order of 1,500

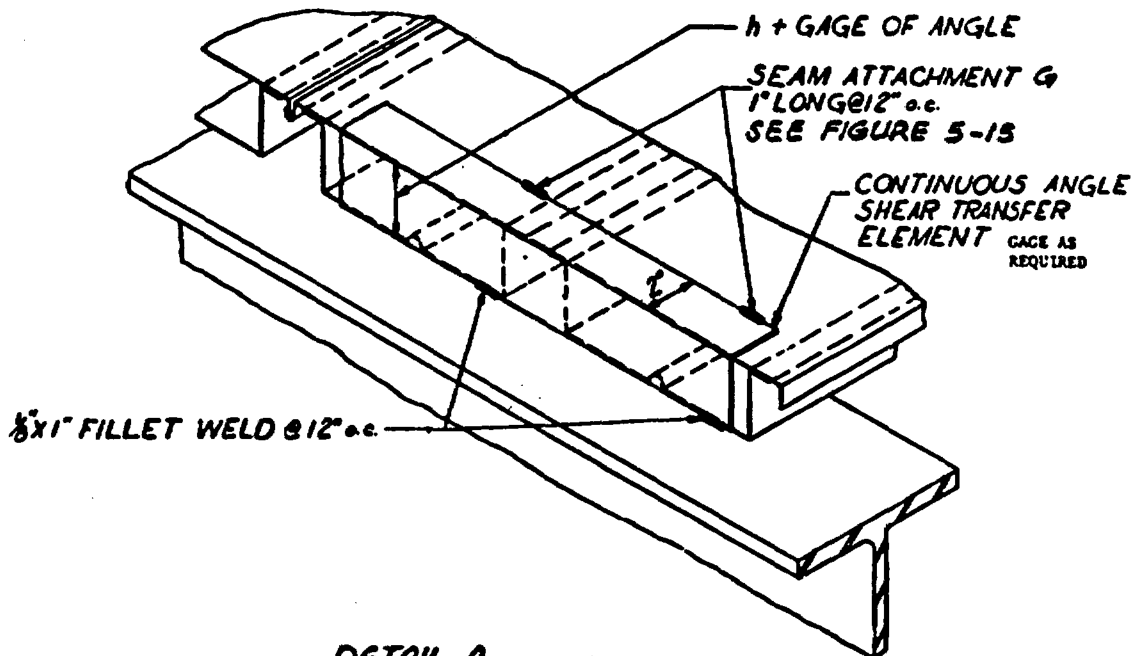


DETAIL A
END CONNECTION TO SUPPORTING BEAMS

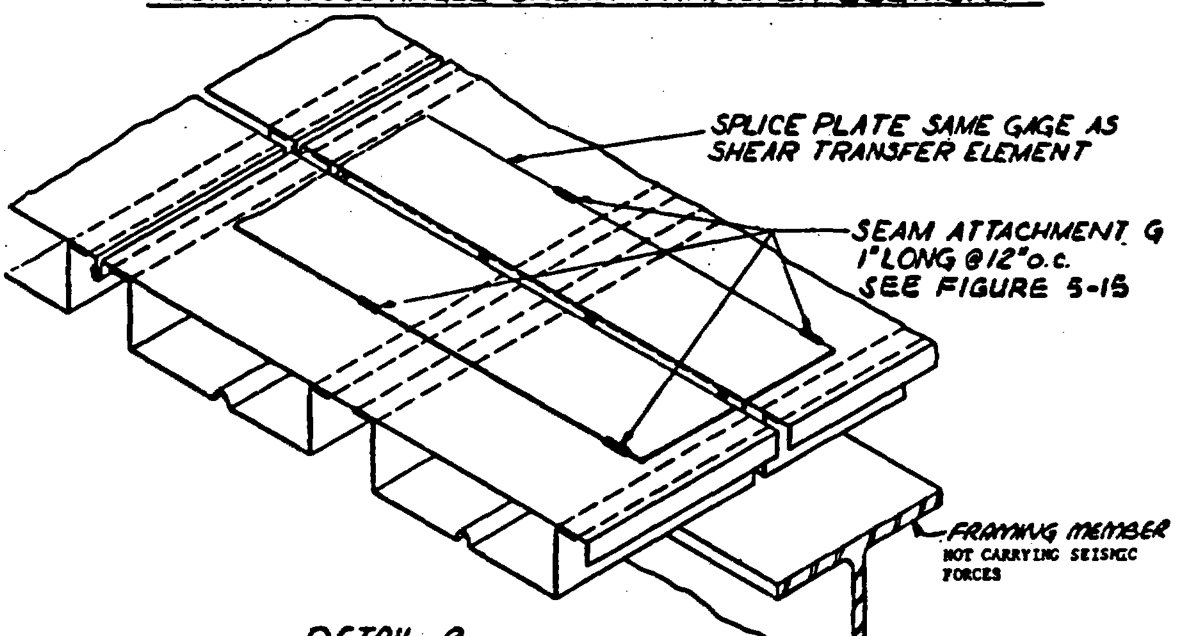


DETAIL B
CONNECTION TO MARGINAL BEAMS

Figure 5-26. Steel Deck Diaphragms - Typical Attachments to Frame for Type B Diaphragms



DETAIL A
CONTINUOUS ANGLE SHEAR TRANSFER ELEMENT



DETAIL B
CONTINUOUS SPLICE PLATE
(SEE FIGURE 5-28 FOR CROSS SECTION)

Figure 5-27. Steel Deck Diaphragms - Typical Attachment of Shear Transfer Elements for Type B Diaphragms

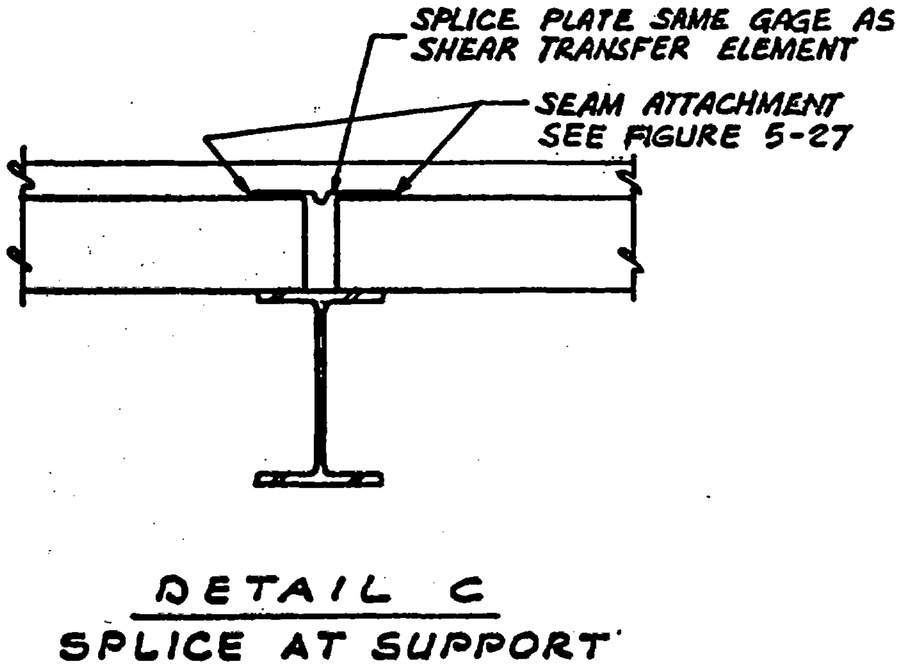
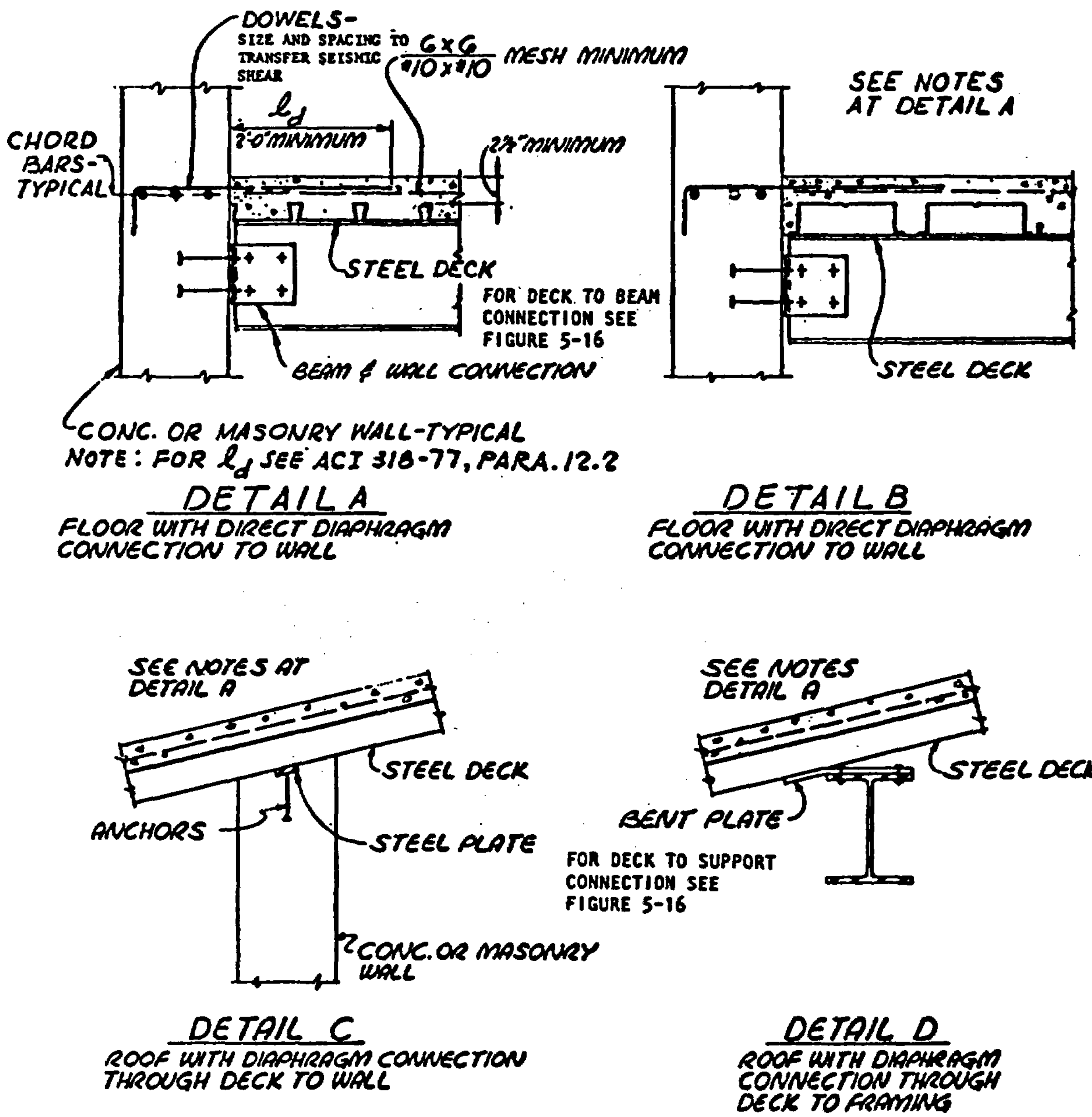


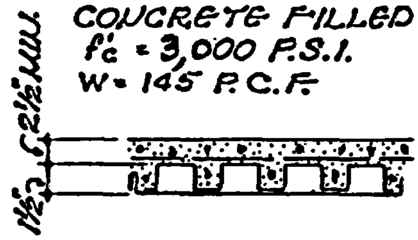
Figure 5-28. Steel Deck Diaphragms - Typical Details Type B Diaphragms



NOTE: WHEN DECKS ARE ATTACHED AT ALL SHEAR TRANSFER POINTS SIMILAR TO DETAILS A AND B, THE DIAPHRAGMS WILL BE DESIGNED IN ACCORDANCE WITH PARAGRAPH 5-4, CONCRETE DIAPHRAGMS. WHEN SHEAR TRANSFER IS THROUGH THE WELDS BETWEEN THE STEEL DECK AND FRAMING, THE DIAPHRAGM WILL BE DESIGNED IN ACCORDANCE WITH PARAGRAPH 5-6c(2), FORMULAS 5-27 and 5-32.

Figure 5-29. Steel Deck Diaphragms - Typical Attachment of Deck with Concrete Fill

TABLE OF ALLOWABLE SHEAR (q_D) AND FLEXIBILTY FACTOR (F)

SECTION	GAGE		SPAN (L_V) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
CONCRETE FILLED f'_c = 3,000 P.S.I. W = 145 P.C.F. 	20-20	q_D	2780	2510	2340	2210	2120	2040	1980
		F	.47	.53	.56	.59	.62	.66	.66
	18-18	q_D	3190	2830	2600	2430	2300	2200	2130
		F	.36	.40	.44	.47	.50	.51	.53
	16-16	q_D	3600	3160	2870	2660	2500	2380	2280
		F	.28	.32	.36	.38	.41	.43	.45
	16-18	q_D	3440	3030	2760	2560	2420	2310	2220
		F	.31	.35	.38	.41	.44	.46	.48

NOTES:
1. BUTTON PUNCH @ 36" o.c.
2. THE GAGES FOR MULTIPLE SHEET DECKS ARE DESIGNATED WITH THE GAGE OF THE FLAT SHEET FIRST AND FLUTED SHEET SECOND.
3. DECK SECTIONS ARE MADE FROM GALVANIZED SHEETS
4. END WELDS CONSIST OF 3 PUDDLE WELDS AT EACH SUPPORT.

Figure 5-30. Steel Deck Diaphragms with Concrete Fill - Allowable Shears and Flexibility Factors

SAMPLE CALCS NO. 6 FOR TYPE A DIAPHRAGM WITH CONC. FILL

$$q_D = q_1 + q_c$$

$$q_1 = \frac{92S(t_1+t_2')K}{bL_v} \qquad K = 1{,}000$$

$$q_c = q_c' + q_c''$$

$$q_c' = \frac{t_f w^{1.5}\sqrt{f_c'}}{200}$$

$$q_c'' = 2\sqrt{\frac{Kb}{d(t_1+t_2')}}$$

$$q_1 = \frac{92\times 1.92(.06+.032)1{,}000}{2\times 6} = 1354.2$$

$$q_c' = \frac{2.5\times 145^{1.5}\sqrt{3{,}000}}{200} = 1195$$

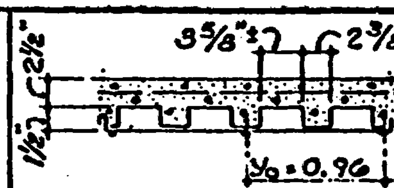
$$q_c'' = 2\sqrt{\frac{1000\times 2}{1.92(.06+.032)}} = 212.8$$

$$q_D = 1354.2 + 1195 + 212.8 = 2762$$

(q_D = 2760 IN FIGURE 5-30 FOR L_v = 6' AND GAGE = 16-18)

$$F = \frac{20\,q_c''}{b^2 q_D} = \frac{20\times 212.8}{2^2\times 2760} = .385 \text{ (SEE FIGURE 5-30)}$$

SAY 38



16-18 GAGE MULTIPLE PLATE DECK WITH 2 1/2" CONC. FILL. 3 END WELDS

$t_f = 2.5"$

$t_1 = .060 \qquad t_2' = 2/3 \times .048 = .032"$

$$S = \frac{\Sigma Y^2}{Y_0} = \frac{2\times .96^2}{.96} = 1.92$$

$I_x = .603$

$b = 2' \qquad h = 1.5"$

$L_v = 6'\text{-}0" \qquad d = 1.92 = 2Y_0$

$n = \frac{4}{2} = 2$

$W = 145$ PCF $\qquad f_c' = 3{,}000$ PSI

Figure 5-31. Steel Deck Diaphragms - Sample Calculation

Table 5-5. Flexibility and Allowable Shears

HORIZONTAL DIAPHRAGMS	F	ALLOWABLE SHEAR Lbs./Lin.Ft. (q_D)
1" Straight Sheathing	1,500	50
2" Straight Sheathing	1,500	40
Conventional 1" Diagonal Sheathing - 1"x6" & 1"x8"	250	300
Conventional 2" Diagonal Sheathing	250	400
Special Construction	75	600

NOTE: THE ALLOWABLE SHEARS SHOWN IN TABLE ARE BASIC VALUES TO WHICH THE FACTORS FOR SPECIES SHOWN IN FIGURE 6-13 WILL BE APPLIED.

and will be considered a very flexible diaphragm. They will not be used for laterally supporting masonry, concrete, or other walls which would be seriously affected by high floor to floor deflection.

(2) *Diagonal sheathing.* The one-third increase usually permitted on working stresses in seismic design is not applicable to the working shears given in this subparagraph.

(a) Conventional construction. These diaphragms will be made up of 1-inch nominal sheathing boards laid at an angle of approximately 45 degrees to supports. Sheathing boards will be directly nailed to each intermediate bearing member with not less than two 8d nails for one-inch by six-inch (1"×6") boards and three 8d nails for boards eight inches (8") or wider, and in addition three 8d nails and four 8d nails will be used for six-inch (6") and eight-inch (8") boards, respectively, at the diaphragm boundaries. End joints in adjacent boards will be separated by at least two joist or stud spaces, and there will be at least two boards between joints on the same support. Boundary members at edges of diaphragms will be designed to resist direct tensile or compressive chord stresses and will be adequately tied together at corners.

1. Conventional wood diaphragms may be used to resist shears not exceeding 300 pounds per lineal foot of width. Two-inch (2") nominal diagonally sheathed diaphragms may be used with a maximum design shear of 400 pounds per lineal foot if 16d common nails are used in lieu of the 8d nails specified for 1 inch nominal sheathing.

2. This category of diaphragms has a value of F of approximately 250 and will be considered as very flexible diaphragms and will not be used to laterally support masonry or concrete walls.

(b) Special construction

1. Special diagonally sheathed diaphragms will include two adjoining layers of 1 inch nominal sheathing boards laid diagonally and at 90 degrees to each other.

2. Special diagonally sheathed diaphragms also include single-layered diaphragms, conforming to conventional construction and which, in addition, will have all elements designed in conformance with the following provision: Each chord or portion thereof may be considered as a beam loaded with a uniform load per foot equal to 50 percent of the unit shear due to diaphragm action. The load will be assumed as acting normal to the chord in the plane of the diaphragm and either toward or away from the diaphragm. The span of the chord, or portion thereof, will be the distance between structural members of the diaphragm, such as joists or blocking, which serve to transfer the assumed [illegible] to the sheathing.

3. Special diagonally sheathed diaphragms may be used to resist shears, due to seismic forces, provided such shears do not stress the nails beyond their allowable safe lateral strength and do not exceed 600 pounds per lineal foot of width. For approximating deflections, a value of F of 75 will be used. Thus they fit into the category of flexible diaphragms.

(3) *Plywood sheathing*

(a) All boundary members will be proportioned and spliced where necessary to transmit direct stresses. Framing members will be at least a 2-inch nominal width. In general, panel edges will bear on the framing members and butt along their center lines. Nails will be placed not less than three-eighths inch (3/8") in from the panel edge, not more than twelve inches (12") apart along intermediate supports and six inches (6") along panel edge-bearings, and will be firmly driven into the framing members. No unblocked panels less than twelve inches (12") wide will be used.

(b) The stiffness of plywood diaphragm webs varies with the thickness of plywood, nailing, and the joint blocking. These variables also occur in the determination of the working shear values of the diaphragm. An F value for determining the sti[illegible] category and for estimating deflections w[illegible] determined using the following formula.

$$F = \frac{33{,}000\, q_{ave}}{q_D^2} \qquad (5\text{-}33)$$

Where

q_D = Allowable shear specified in table 5-6 in pounds per foot.

(c) For plywood diaphragms the tabular values of q_D vary between 110 pounds per foot to 820 pounds per foot. From this, the value of F can be determined as varying between 300 and 20. Thus, plywood diaphragms can be very flexible, flexible, or semi-flexible diaphragms depending on the selection of the type of diaphragm to be used.

(d) Nailing. Pneumatically or mechanically driven steel wire staples with a minimum crown width of 7/16 inch is an acceptable alternate method of attaching diaphragms. The crown of the staple will be installed parallel to the framing member.

Common wire nail	*Staple*	*Minimum staple penetration in framing member*
6d	No. 14 gage	1 inch
8d	No. 13 gage	1 inch
10d	No. 12 gage	1-1/8 inch

Table 5-6.

Recommended Shear in Pounds per Foot for Horizontal Plywood Diaphragms with Framing of Douglas Fir, Larch or Southern Pine (a) for Wind or Seismic Loading

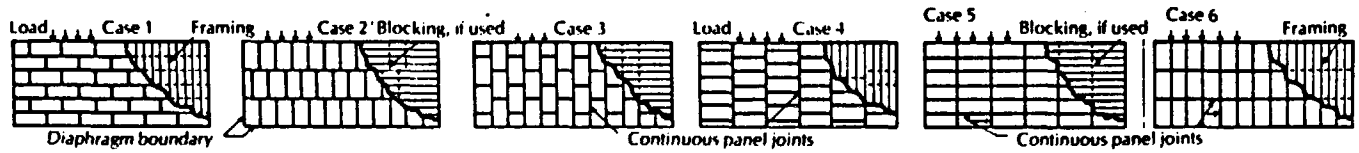
Grade	Common Nail Size	Min. Nail Penetration in Framing (inches)	Minimum Plywood Thickness (inch)	Min. Width of Framing Member	Blocked Diaphragms: Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4,) and at all panel edges (Cases 5 & 6) (b): 6 / Nail spacing (in.) at other plywood panel edges. (Cases 1, 2, 3 & 4): 6	4 / 6	2½ / 4	2 / 3	Unblocked Diaphragms: Nails Spaced 6" Max. at Supported Edges (b): Case 1 (No Unblocked Edges or Continuous Joints Parallel to Load)	All Other Configurations (Cases 2, 3, 4, 5 & 6)
STRUCTURAL I C-D INT-APA or STRUCTURAL I C-C EXT-APA	6d	1-1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1-5/8	1/2	2	320	425	640 (c)	730 (c)	285	215
				3	360	480	720	820	320	240
C-D INT-APA, STRUCTURAL II C-D INT-APA, STRUCTURAL II C-C EXT-APA, and other APA grades except Species Group 5	6d	1-1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d	1-1/2	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			1/2	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d	1-5/8	1/2	2	290	385	575 (c)	655 (c)	255	190
				3	325	430	650	735	290	215
			5/8	2	320	425	640 (c)	730 (c)	285	215
				3	360	480	720	820	320	240

(a) For framing other species: (1) Find species group of lumber in Table 8.1 A, NFPA 1977 Nat'l Design Spec. (2) Find shear value from table for nail size, and for Structural plywood (regardless of actual grade). (3) Multiply value by 0.82 for Lumber Group III or 0.65 for Lumber Group IV.

(b) Space nails 12 in. on center along intermediate framing members for roofs, and 10 inches on center for floors..

(c) Reduce tabulated allowable shears 10 percent when boundary members provide less than 3-inch nominal nailing surface.

Notes: Design for diaphragm stresses depends on direction of continuous panel joints with reference to load, not on direction of long dimensions of plywood sheet. Continuous framing may be in either direction for blocked diaphragms.



NOTE: Table 5-6 is reprinted, with permission, from Table 32 in PLYWOOD CONSTRUCTION GUIDE, © 1978 American Plywood Association.

b. Typical Details. Refer to figures 5-32 through 5-35.

5-8. Horizontal bracing (wood or steel). *a. General Design Criteria.* The criteria used to design horizontal steel bracing will be the "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," AISC. The criteria for wood bracing will be "National Design Specification for Wood Construction." Reference should be made to chapter 3, paragraphs 3-3(J)1g and 3-3(J)2d; paragraphs 5-2*a*(2) and 5-3d; and chapter 6, paragraph 6-7, where applicable.

b. General Discussion

(1) *General system.* The entire system must be as simple, direct, positive, and effective as practicable. Although it is ordinarily preferable in nonseismic design to have one definite, predetermined, and adequate means of resisting any given load seismic purposes, when the damage to a sp truss, column, or other member could cause complete failure, multiple systems are generally used. For example, if one truss is damaged, these braces would pick up its load sufficiently to prevent complete collapse.

(2) *Functions of roof and floor bracing.* The basic functions of roof or floor bracing are to: (a) keep the top (compression) chords of trusses (or frames) from buckling laterally, (b) prevent trusses from tipping over, (c) steady the columns, and (d) transmit the lateral forces to the vertical bracing system.

(3) *Connections.* In lieu of developing the full capacity of the member or part concerned, the connections will be designed for 1.25 times the design force without the one-third increase usually permitted.

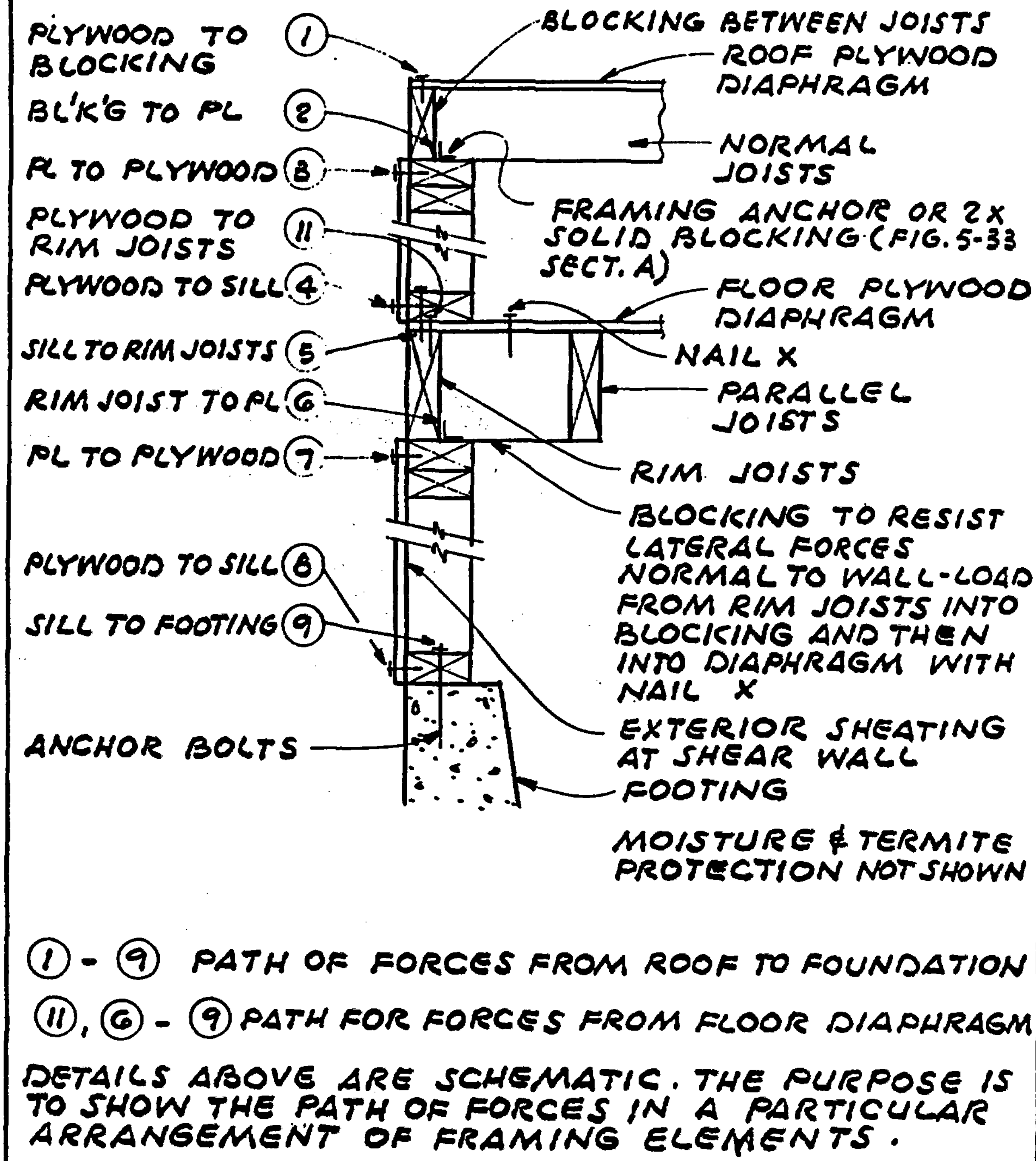


Figure 5-32. Wood Diaphragm and Shear Wall Nailing

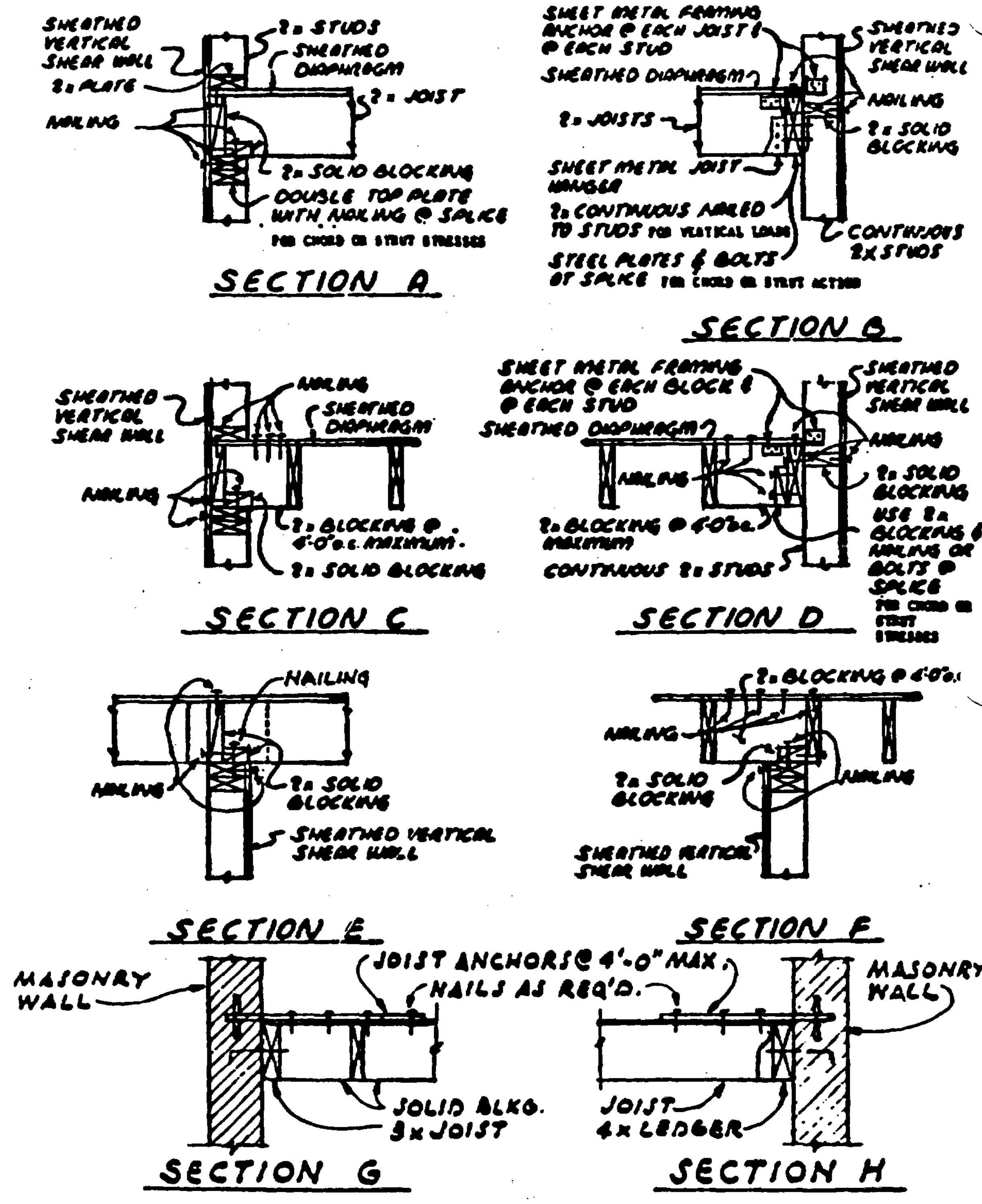


Figure 5-33. Wood Diaphragms - Typical Connection Details

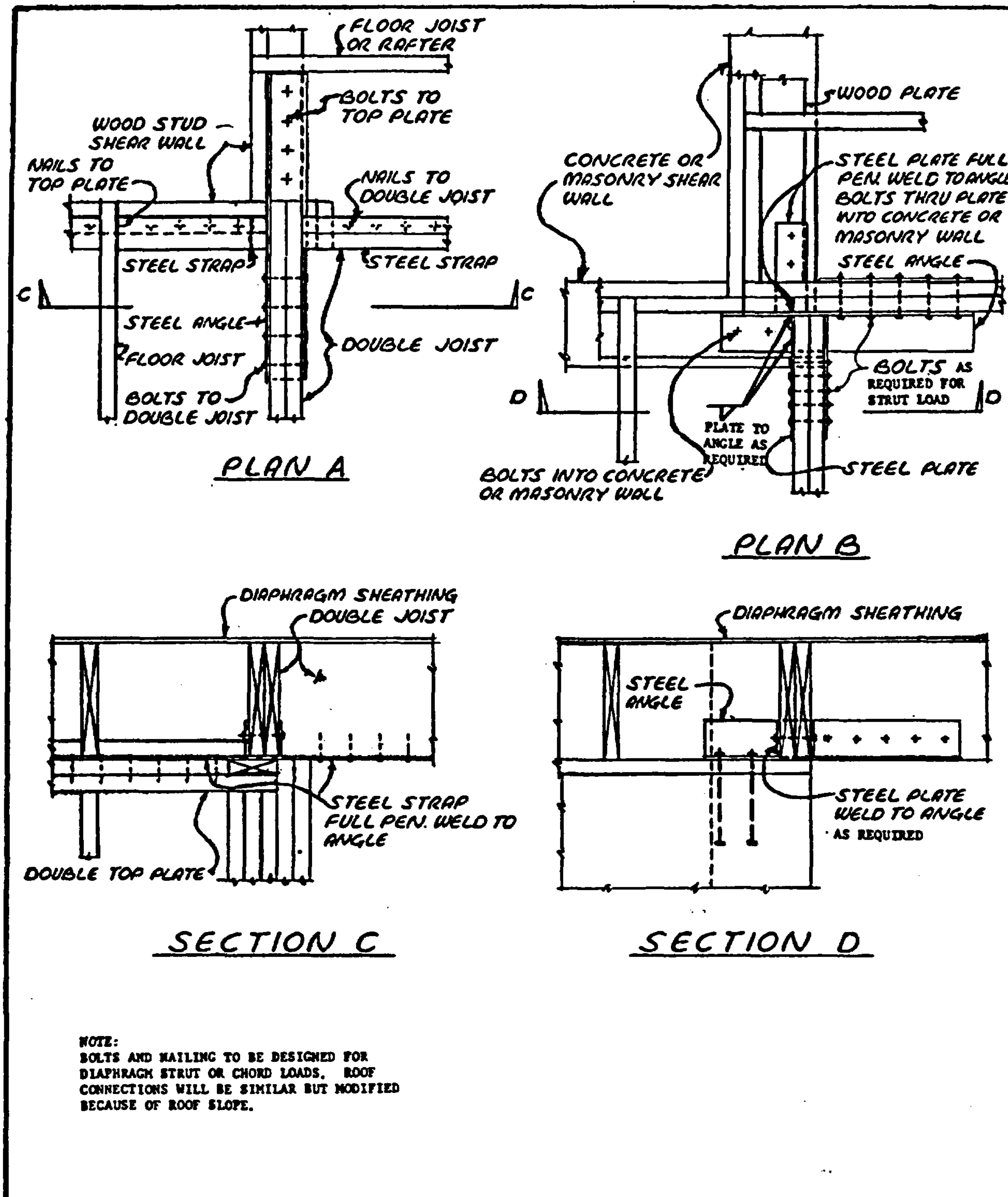
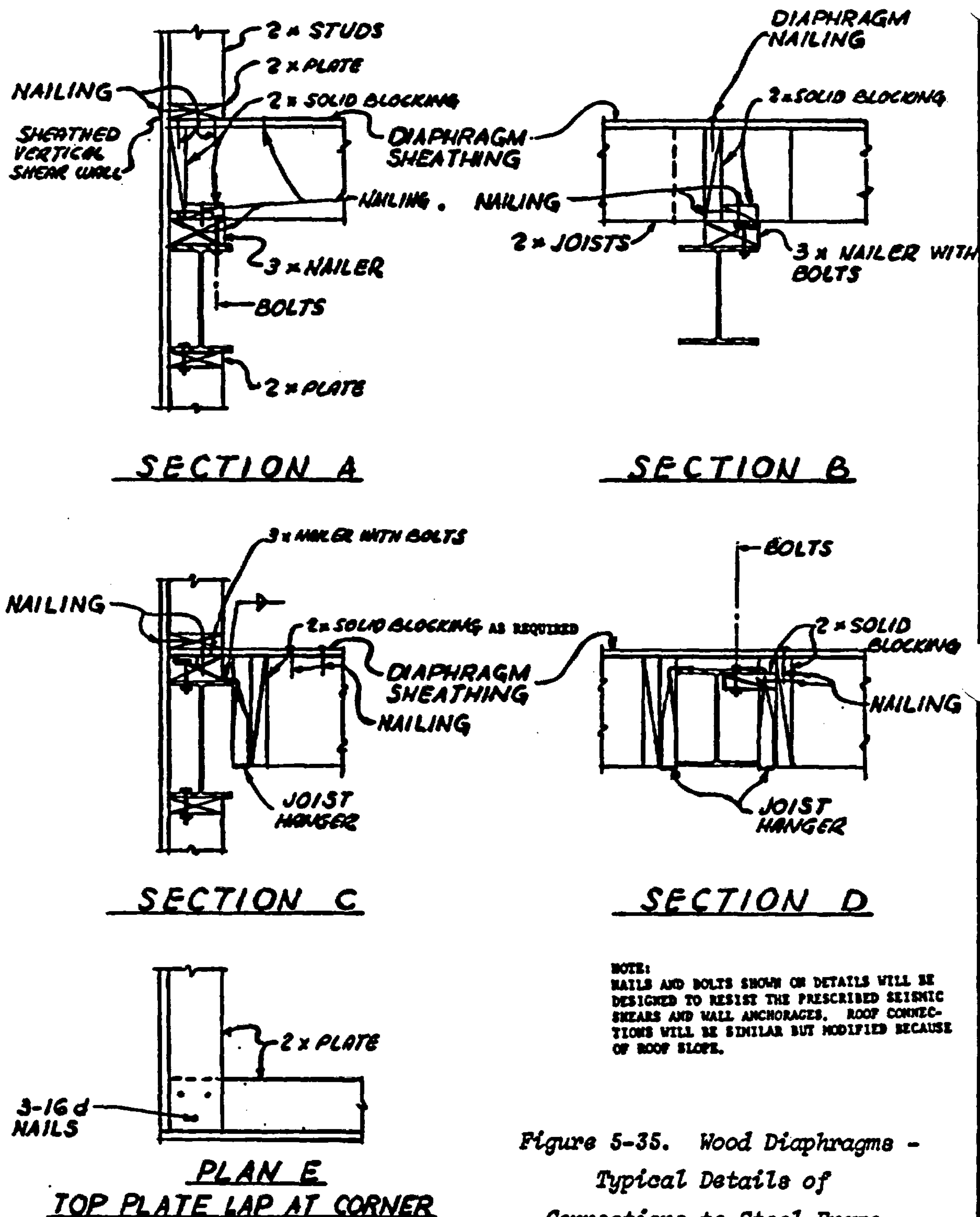


Figure 5-34. Wood Diaphragms - Typical Strut Connection Details



NOTE:
NAILS AND BOLTS SHOWN ON DETAILS WILL BE DESIGNED TO RESIST THE PRESCRIBED SEISMIC SHEARS AND WALL ANCHORAGES. ROOF CONNECTIONS WILL BE SIMILAR BUT MODIFIED BECAUSE OF ROOF SLOPE.

Figure 5-35. Wood Diaphragms - Typical Details of Connections to Steel Frame

CHAPTER 6
WALLS AND BRACED FRAMES

6-1. Purpose and scope. This chapter prescribes the criteria for the design of walls and vertical bracing of buildings for seismic resistance; indicates the principles and factors governing the application of horizontal forces normal to the plane of walls, parallel to the plane of walls (shear walls), and parallel to the plane of braced frames; gives certain design data; and illustrates typical details of construction.

6-2. General. Buildings are composed of vertical and horizontal structural elements which resist lateral forces. The forces originating from the mass of vertical elements may be transferred either directly to the ground, as in the case of vertical cantilevers, or to horizontal resisting elements other than the ground through vertical beam action of the vertical elements. The forces originating from the mass tributary to horizontal elements are distributed by such horizontal elements to vertical elements which in turn transmit such forces to the ground. Vertical elements used to transfer lateral forces to the ground are: (1) shear walls, (2) braced frames, and (3) moment resisting frames. This paragraph covers basic functions, essential characteristics, and seismic loads for walls (loaded normal and parallel to their plane) and braced frames. Specific factors, criteria, and typical details of design of walls and braced frames using various materials of construction are described in paragraphs 6-3 through 6-8. Moment resisting frames are covered in chapter 7.

a. Types of Walls and Loading Conditions. Walls may be subjected to both vertical (gravity) and horizontal (wind or earthquake) forces. A wall carrying a vertical load other than its own weight is called a bearing wall. The horizontal forces acting on a wall may be either normal to the wall or parallel to the wall. A shear wall resists horizontal forces parallel to the wall. Any wall or partition which carries a vertical load other than its own weight, and/or which resists a horizontal force parallel to the wall, is classified as a structural wall. The combined effects of horizontal forces and vertical load on a wall must be considered. Walls and partitions must be designed to withstand all vertical loads and horizontal forces, both parallel to and normal to the flat surface, with due allowance for the effect of any eccentric loading or overturning forces generated. Any wall which is isolated on 3 sides (both ends and top) so as not to resist external loads or forces parallel to the wall is classified as nonstructural. A nonstructural wall shall be able to resist horizontal wind or seismic forces normal to the wall. Nonisolated walls will obviously participate in shear resistance to horizontal forces parallel to the wall, since they tend to deflect and be stressed when the framework or horizontal diaphragms deform under lateral forces.

b. Loads Normal to Walls. Walls and partitions must safely resist horizontal seismic forces normal to their flat surface (figs 6-1 and 6-3 and fig 4-5); and moments and shears induced by relative deflections of the diaphragms above and below (fig 6-2). For diaphragm deflections refer to chapter 5. When a wall resists horizontal forces perpendicular to it, it usually distributes such loads vertically to the horizontal resisting elements above or below. It may also distribute horizontally to shear walls or frames (chap 4, para 4-4*d* and fig 4-5). A wall may be either continuous or discontinuous across its supports. The horizontal seismic force normal to a wall is a function of its weight. The formula given in chapter 3, paragraph 3-3(G), for the magnitude of this force is $F_p = ZIC_pW_p$ with $C_p = 0.30$. (For cantilevered walls, see paragraph *c* below.) This seismic force will be applied to the wall in both inward and outward directions. However, wind forces, other forces, or interstory drift will frequently govern the design.

c. Cantilevered Walls. Where walls, such as parapets, are cantilevered, the anchorage for reaction and cantilever moment is required to be fully developed (fig 6-3). C_p for this condition is 0.80 per chapter 3, paragraph 3-3(G) and table 3-4. Where a parapet wall is anchored to a concrete roof slab and is not a continuation of a wall below, the roof slab will be designed for the cantilever moment. Where the parapet is a continuation of a wall below, the cantilever moment will be divided between the concrete slab and the wall below in proportion to their relative stiffnesses. Where the parapet is an extension of a wall below and is anchored to a roof or floor of wood, metal deck, or other similar materials, the moment at the base of the parapet will be developed into the wall below. In this case the anchorage force to the roof will be determined by the usual methods of analysis, assuming a pinned condition for the connection of the roof to the wall.

d. Shear Walls—Loads Parallel to Wall.

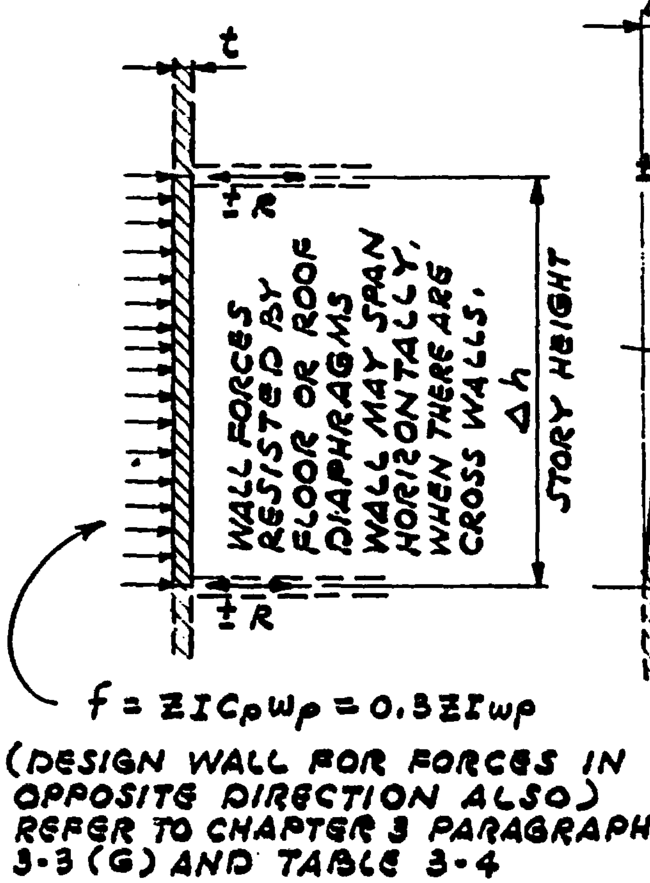


Figure 6-1. Load Normal to Wall

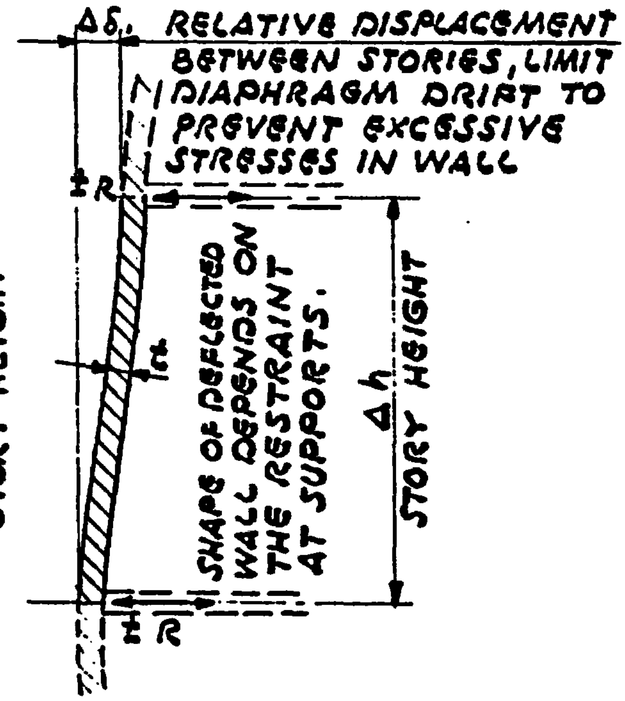


Figure 6-2. Deflections Induced by Relative Deflections of Diaphragms (Refer to Chapter 5)

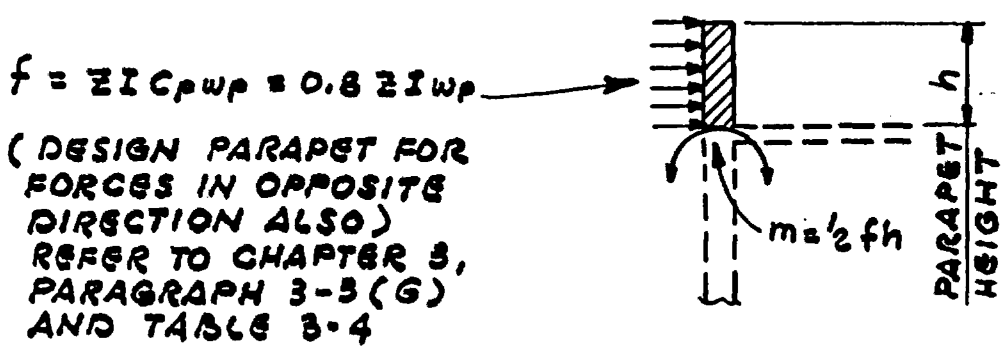


Figure 6-3. Parapet Loading

Horizontal forces at any floor or roof level are generally transferred to the ground (foundation) by using the strength and rigidity of shear walls (and partitions). A shear wall may be considered analogous to a cantilever plate girder standing on end in a vertical plane where the wall performs the function of a plate girder web, the pilasters or floor diaphragms function as web stiffeners, and the integral reinforcement of the vertical boundaries function as flanges. Axial, flexural, and shear forces must be considered in the design of shear walls. The tensile forces on shear wall elements resulting from the combination of seismic uplift forces and seismic overturning moments must be resisted by anchorage into the foundation medium unless they can be overcome by gravity loads (e.g., 0.9 of dead load) mobilized from neighboring elements (this is discussed more fully in chap 4, para 4-4*b*, 4-4*c*(2), and 4-8). A shear wall may be constructed of materials such as concrete, wood, unit masonry, or metal in various forms. Working stresses of such materials as cast-in-place reinforced concrete and reinforced unit-masonry are well known and present no problem to the designer once the loading and reaction system is determined. Other materials frequently used to support vertical loads from floors and roofs have well-established vertical load-carrying characteristics but have required tests to demonstrate their ability to resist lateral forces. Various types of wood sheathing and metal siding fall into this category. Where a shear wall is made up of units such as plywood, gypsum wallboard, tilt-up concrete units, or metal panel units, its characteristics are, to a large degree, dependent upon the attachments of one unit to another and to the supporting members.

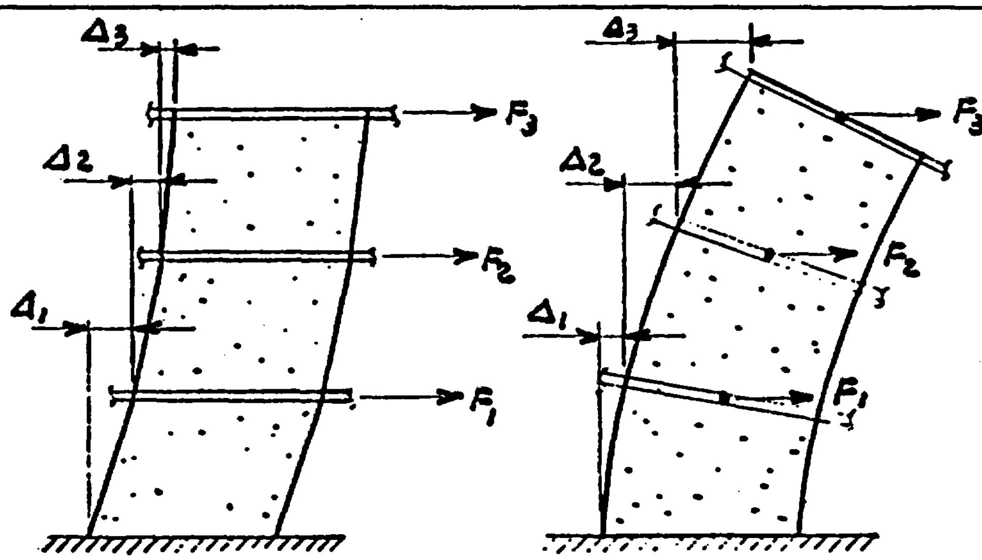
(1) *Rigidity.* The magnitude of the total lateral forces at any story or level depends upon the structural system as a whole. The proportion of that total horizontal load carried by a particular shear wall is based on its relative rigidity considering the rigidity of the other walls and the diaphragms. The rigidity of a shear wall is inversely proportional to its deflection under a unit horizontal force. Where shear walls are tied together by a rigid diaphragm or bracing so that all must deflect equally, the total translational lateral force is shared in direct proportion to their relative rigidities (torsional moments must also be considered, chap 4, para 4-4*e*(2)). Wall deflection is the sum of the deformations due to shear and flexure (fig 6-4) plus any additional displacement that may occur due to rotation at the base.

(a) The rotation at the foundation can greatly influence the overall rigidity of a shear wall because of the very rigid nature of the shear wall itself; however, the rotational influence on relative rigidities of walls for purposes of horizontal force distribution may not be as significant. Considering the complexities of soil behavior, a quantitative evaluation of the foundation rotation is generally not practical, but a qualitative evaluation, recognizing the limitations and using good judgment, will be provided.

(b) The relative rigidity of concrete or unit masonry walls with normal openings is usually much greater than that of any building framework. Thus, the walls tend to resist essentially all or a major part of the lateral force.

(2) *Shear wall with openings.* The impact on the size and number of openings in shear walls to resist lateral forces must be considered. If openings are very small, their effect on the overall state of stress in a shear wall is minor. Large openings have a more pronounced effect and, if large enough, result in a system in which typical frame action predominates. Openings normally occur in regularly spaced vertical rows throughout the height of the wall and the connection between the wall sections is provided by either connecting beams (or spandrels) which form a part of the wall, or floor slabs, or a combination of both. If the openings do not line up vertically and/or horizontally, the complexity of the analysis is greatly increased. In most cases, a rigorous analysis of a wall with openings is not required. When designing a wall with openings, the deformations must be visualized in order to establish some approximate method to analyze the stress distribution to the wall. Figures 6-4 and 6-5 give some visual descriptions of such deformations. The major points that need to be considered are: (1) the lengthening and shortening of the extreme sides (boundaries) due to deep beam action, (2) the stress concentration at the corner junctions of the horizontal and vertical components between openings, and (3) the shear and diagonal tension in the horizontal and vertical components.

(a) Relative rigidities of piers and spandrels. The ease of methods of analysis for walls with openings is greatly dependent on the relative rigidities of the piers and the spandrels, as well as the general geometry of the building. Figure 6-6 shows two extreme examples of relative rigidities of exterior walls of a building. In figure 6-6a the piers are very rigid and the spandrels are very flexible. Assuming a rigid base, the shear walls act as vertical cantilevers. When a lateral force is applied, the spandrels act as struts which flexurally deform to be compatible with the deformation of the cantilever piers. It is relatively simple to determine the forces on the cantilever piers by ignoring the deformation



(a) Shear Deformation (b) Flexural Deformation

Figure 6-4. Shear Wall Deformation

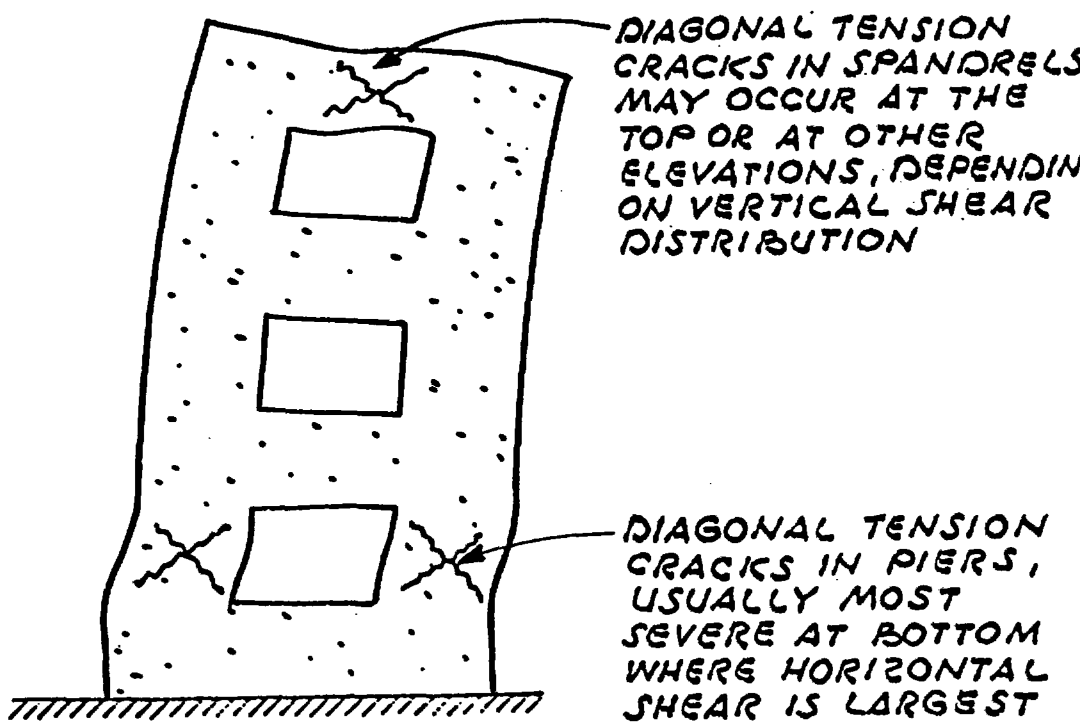


Figure 6-5. Deformation of Shear Wall with Openings

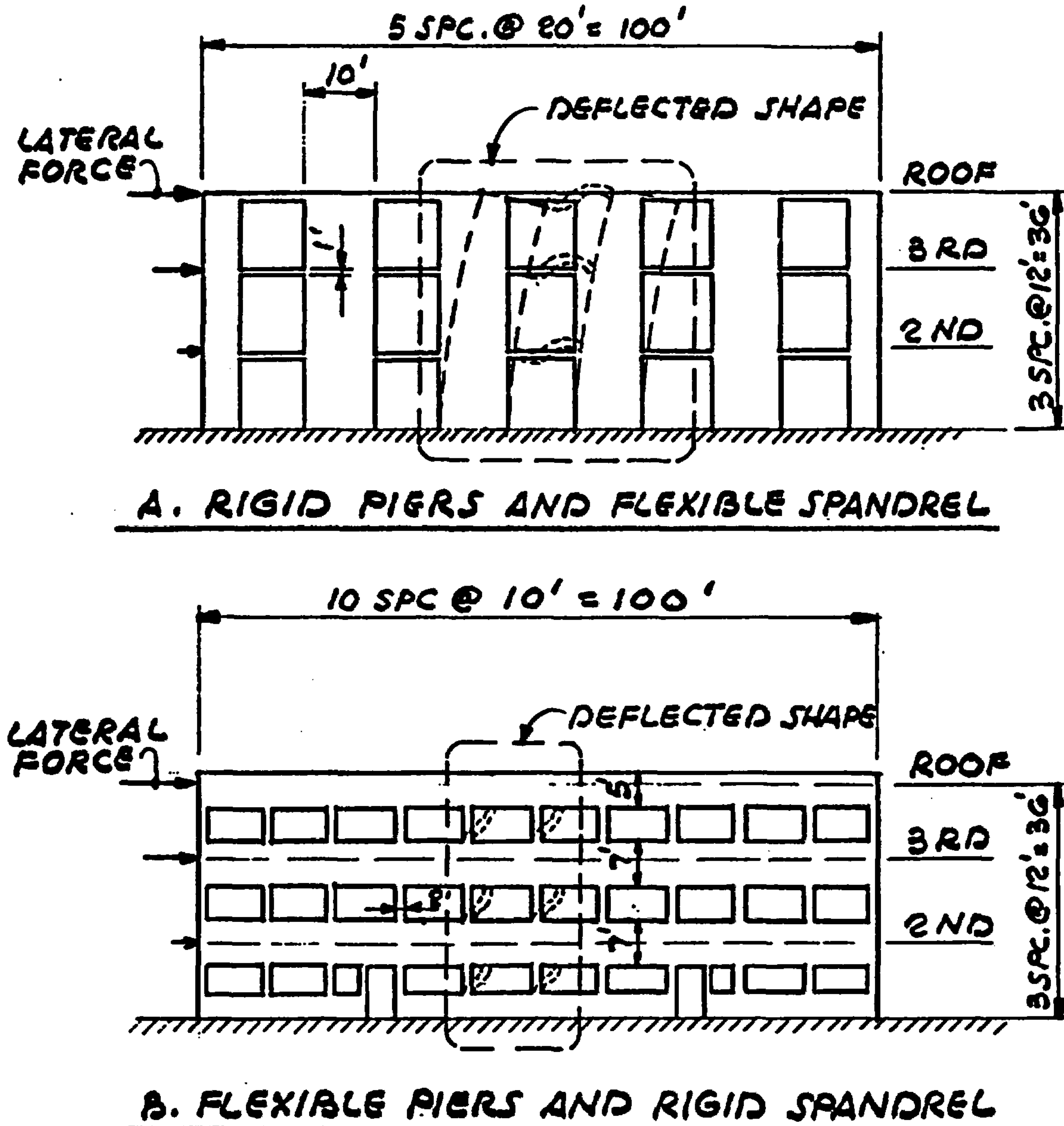


Figure 6-6. Relative Rigidities of Piers and Spandrels

characteristics of the spandrels. The spandrels are then designed to be compatible with the pier deformations. In figure 6-6b, the piers are relatively flexible compared to the spandrels. The spandrels are assumed to be infinitely rigid and the piers are analyzed as fixed ended columns. The spandrels are then designed for the forces induced by the columns. The overall wall system is also analyzed for overturning forces that induce axial forces into the columns. The calculations of relative rigidities for both cases shown in figure 6-6 can be aided by the charts in figure 6-11, paragraph 6-3*b*(3). For cases of relative spandrel and pier rigidities other than those shown, the analysis and design becomes more complex.

(b) Methods of analysis. Approximate methods for analyzing walls with openings are generally acceptable. (See app C, example C-4.) For the simple cases shown in figure 6-6 the procedure is straightforward. For more complex cases, a variation of assumptions may be used to determine the most critical loads on various elements, thus resulting in a conservative design. (*Note:* In some cases a few additional reinforcing bars, at little additional cost, can greatly increase the strength of shear walls with openings.) However, when the reinforcement requirements or the resulting stresses of this approach appear excessively large, a rigorous analysis may be justified.

(3) *Dual systems.* Buildings may utilize both shear walls and moment resisting space frames to resist lateral forces. The total lateral load is assumed to be resisted by the shear walls and the frame is assigned to resist nominally 25 percent of the total lateral load. It is assumed that the contribution of the frame for lateral resistance will provide redundancy and will provide a reserve strength against complete collapse if the shear walls should fail. However, the difference in behavior between walls and frames results in non-uniform interacting forces between these elements when they are connected together by floor slabs (see chap 4, para 4-4*e*(3) and fig 4-7). Therefore, the distribution of forces in accordance with the relative rigidities and the interaction of walls and frames must also be considered (table 3-3).

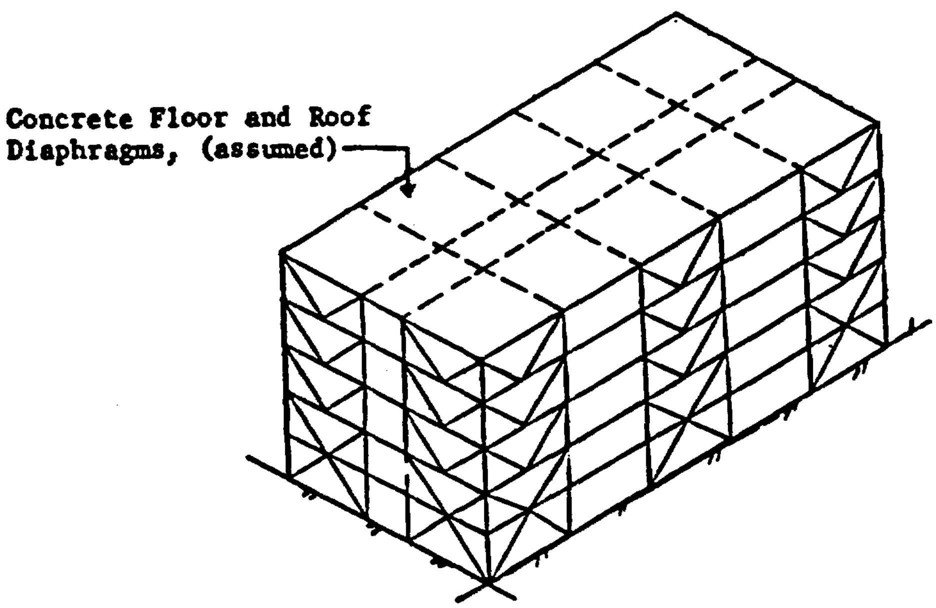
(4) *Special loading and detail requirements.* All portions of a shear wall will be designed to resist the combined effects of axial loads (if any) and other boundary forces as determined from a rational distribution of the total prescribed lateral forces on the structure as a whole. Special criteria to control brittle behavior and to provide greater elastic response capacity of shear walls in concrete and unit-masonry are required as stipulated in paragraph 6-3 and in chapter 3, paragraph 3-3(J)1h, respectiv A modified load factor for shear and diagonal sion is used for buildings without a 100 percent ductile moment resisting space frame. Vertical boundary elements (e.g., structural steel or confined reinforcement) are to be provided at the edges of shear walls (and similar confinement adjacent to wall openings) under certain prescribed conditions (para 6-3*a*(1)(d) and 6-8).

e. Braced Frames. The use of braced frames is an acceptable alternative method to resist lateral forces in place of shear walls. The material may be reinforced concrete, structural steel, or wood. Vertical bracing systems are used to transfer the horizontal forces at the floor or roof levels to the foundations. The function of the bracing is to resist forces that tend to deform the building in the direction parallel to the plane of that bracing, and to transmit these lateral loads to the foundation. As with other systems, the deformations to be expected in a major earthquake can be much greater than those found using the prescribed forces. As the ductility of conventional braced systems has not been adequately demonstrated, multiple braces (see fig 6-7) should be used whenever possible to increase the redundancy. See paragraph 6-7 for vertically braced frames.

(1) *Layout.* When planning a bracing syste a building, consider the structure as a whole (see fig 5-4 and 6-7; also, refer to chap 5, para 5-2*a*(2), for horizontal bracing systems). Visualize the ways in which a structure might fail, and provide bracing to keep the structure from collapsing. The designers must be certain just where every door, window, passageway, obstruction, and other controlling features will be located before placing the bracing. The architect must be certain just where the bracing is to be placed before deciding the type of fenestration.

(2) *Lateral force resistance.* The braced framing must be designed to carry the lateral force reactions from the roof and floors. The entire system must be as simple, direct, positive, and effective as practicable. However, multiple systems will generally be used for seismic purposes when the damage to a specific member could cause complete failure. For example, if one braced frame should be damaged, the other braced frames would pick up its load sufficiently to prevent complete collapse. Locate vertical braced frames so as to limit torsion.

6-3. Cast-in-place concrete shear walls and concrete braced frames. *a. General Design Criteria.* The criteria used to design reinforced crete shear walls will be ACI 318-77 except Ap dix A, and as modified by the SEAOC Section 3 (re-



(NOTE: See Figure 5-4 for system with horizontal bracing system.)

Figure 6-7. Bracing for A Tier Building printed below) and in this manual. For tilt-up and other precast concrete shear walls, refer to paragraph 6-4.

(1) SEAOC Section 3, Concrete Shear Wall and Braced Frames[a] *(Modifications are in italics).*

(A) General.

Design and construction of reinforced concrete shear walls and reinforced concrete braced frames used to resist seismic forces shall conform to the requirements of the A.C.I. Building Code, A.C.I. 318, and all the requirements of *SEAOC* Section 3 *as modified herein.*

Shear walls and braced frames shall be designed by the strength design method except that the alternate design method may be used provided that the factor of safety in shear and diagonal tension is equivalent to that achieved with the strength design method.

A.C.I. 318, for earthquake loading, shall be modified to:

$$U = 1.4(D+L) + 1.4E \qquad (6\text{-}1)^{b}$$

$$U = 0.9D + 1.4E \qquad (6\text{-}2)$$

provided further than 2.0 E shall be used in both equations in calculating shear and diagonal tension in buildings other than those complying with requirements for buildings with K = 0.67.

(B) Braced Frames.

Reinforced concrete members of braced frames subjected primarily to axial stresses shall have special transverse reinforcing as set forth in Section 2(E)4[c] throughout the full length of the member. Tension members shall additionally meet the requirement for compression members.

EXCEPTION: *In Zone 1 and for Zone 2 buildings under 160 feet, the provisions of chapter 7, paragraphs 7-4a(15) and (16) will satisfy this requirement.*

(C) Shear and Diagonal Tension Strength Design.

1. Shear Stress. The nominal ultimate shear stress v_u, resulting from forces acting parallel to shear walls shall be computed by

$$v_u = \frac{V_u}{\phi A_c} \qquad (6\text{-}3)$$

where

V_u = Ultimate shear computed according to Section 1 and including the effect of gravity loads.

A_c = Area of concrete sections resisting V_u.

2. Shear Stress Limits. The ultimate shear stress v_u thus computed shall not exceed that given by

$$v_u = 2\sqrt{f'_c} + pf_y, \qquad (6\text{-}4)$$

where "p" is the ratio of the area of reinforcement to the area of concrete

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[a]From the publication "Recommended Lateral Force Requirements and Commentary" by the Seismology Committee, Structural Engineers Association of California. Copyright 1976, Structural Engineers Association of California, and reproduced with permission.

[b]Formulas have been renumbered such that SEAOC Formula 3-1 is designated as 6-1 in this manual.

[c]SEAOC Section 2, Concrete Ductile Moment Resisting Space Frames, is reprinted, as modified in this manual, as chapter 7, paragraph 7-3a(1)(e.g., Section 2(E)4 is paragraph 7-3a(1)(E)4).

section resisting the shear V_u. At least an equal percentage of reinforcement "p" shall be provided perpendicular to that required to satisfy Formula (6-4).

The average horizontal shear v_u for all wall piers sharing a common lateral force component shall not exceed

$$8\sqrt{f'_c} \tag{6-5}$$

and the v_u in any of the individual wall piers shall be not more than

$$10\sqrt{f'_c} \tag{6-6}$$

The value of the vertical shear v_u shall not exceed

$$10\sqrt{f'_c} \tag{6-7}$$

for horizontal wall elements.

3. Minimum Reinforcement. The minimum reinforcing ratio "p" for all walls designed to resist seismic forces acting parallel to the wall shall be 0.0025 each way. The maximum spacing of reinforcement each way shall not exceed d/3 or eighteen inches (18"), whichever is smaller, where "d" is the dimension of the wall element parallel to the shear force. That portion of the wall reinforcement required to resist design shears shall be uniformly distributed. *See figure 6-8.*

4. Anchorage of Reinforcement. Wall reinforcement required to resist wall shear shall be terminated with not less than a 90 degree bend plus a 6 bar diameter extension beyond the boundary reinforcing at vertical and horizontal end faces of wall sections. Wall reinforcement terminating in boundary columns or beams shall be fully anchored into the boundary elements.

(D) Vertical Boundary Members for Shear Walls. *(See figure 6-9)*

Special vertical boundary elements shall be provided at the edges of concrete shear walls *designated as Shear Wall Type A in chapter 3, table 3-7.*[d] These elements shall be composed of concrete encased structural steel elements of ASTM, A36, *A441, A500 (Grades B and C), A501, A572 (Grades 42, 45, 50 and 55) or A588 or shall be concrete reinforced as required for columns in Section 2(E) with special transverse reinforcement as described in Section 2(E)4 for the full length of the element. The longitudinal reinforcing in these concrete boundary elements shall conform to the requirements of Section 2(C)2.*[c] *(i.e., chap 7, para 7-3a*(1)(C)2).[e]

The boundary vertical elements and such other similar vertical elements as may be required shall be designed to carry all the vertical stresses resulting from the wall loads in addition to tributary dead and live loads and from the horizontal forces as prescribed in *chapter 3.* Horizontal reinforcing in the walls shall be fully anchored to the vertical elements.

Similar confinement of horizontal and vertical boundaries at wall openings shall also be provided unless it can be demonstrated that the unit compressive stresses at the opening are less than the prescribed limits when using Formulas (6-1) and (6-2) modified with 2.0E instead of 1.4E.

[d]In Zones 2, 3, and 4 this includes K > 1.0 buildings over 80 feet in height and all K = 0.8 buildings. In Zone 1, this includes K = 0.8 buildings over 80 feet in height.

[e]1980 SEAOC Revisions.

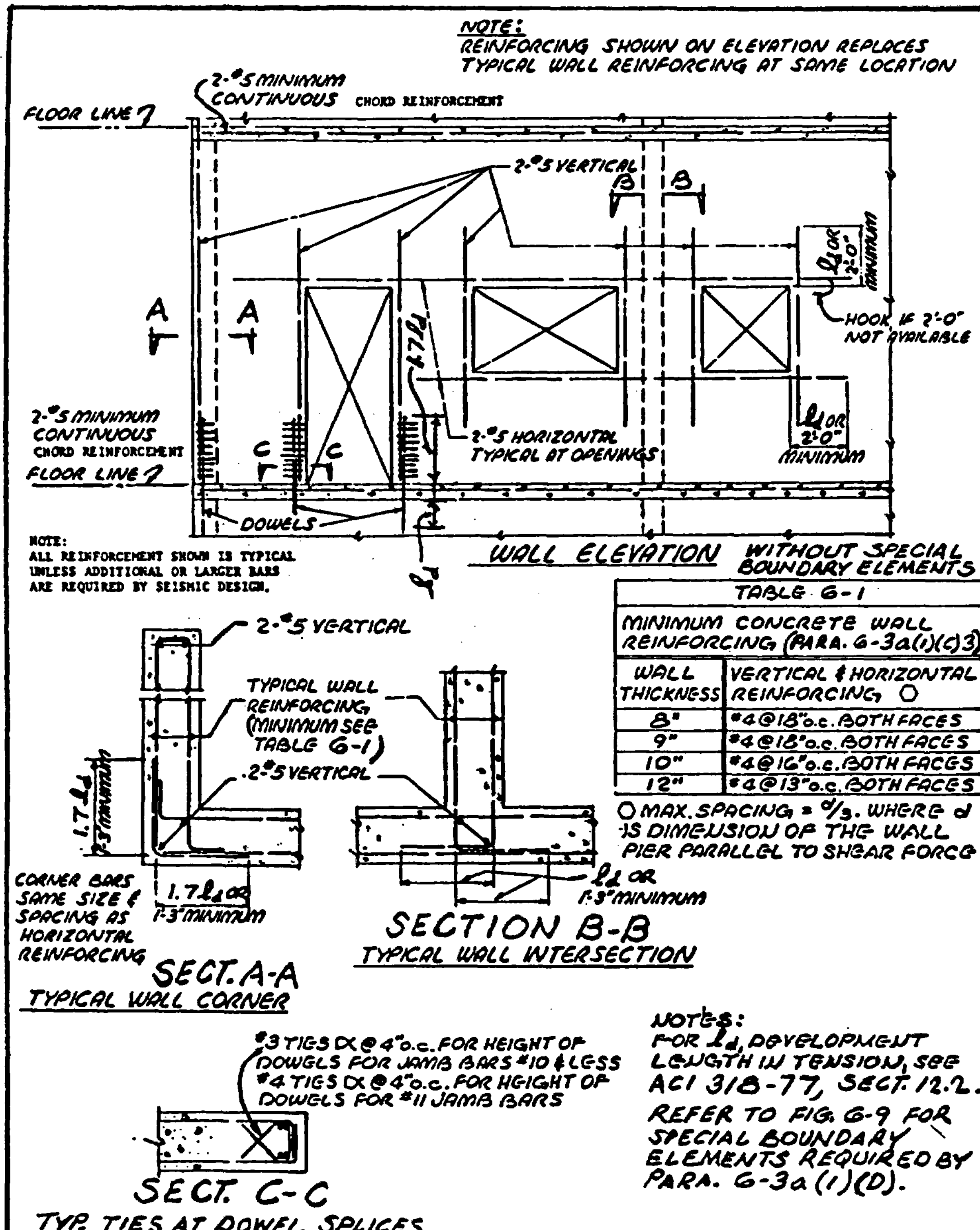


TABLE 6-1

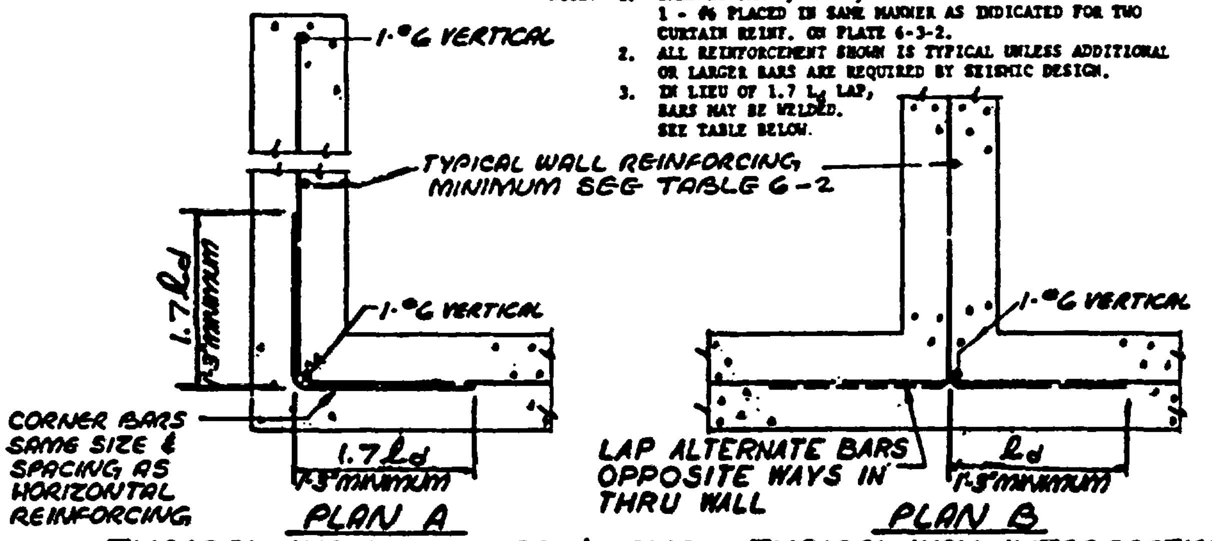
MINIMUM CONCRETE WALL REINFORCING (PARA. 6-3a(1)(c)3)

WALL THICKNESS	VERTICAL & HORIZONTAL REINFORCING ○
8"	#4@18" o.c. BOTH FACES
9"	#4@18" o.c. BOTH FACES
10"	#4@16" o.c. BOTH FACES
12"	#4@13" o.c. BOTH FACES

○ MAX. SPACING = d/3, WHERE d IS DIMENSION OF THE WALL PIER PARALLEL TO SHEAR FORCE

Figure 6-8A Minimum Concrete Shear Wall Reinforcement (Two curtains)

NOTE:
1. BARS AT JAMBS, HEADS, AND SILLS OF OPENINGS WILL BE 1 - #6 PLACED IN SAME MANNER AS INDICATED FOR TWO CURTAIN REINF. ON PLATE 6-3-2.
2. ALL REINFORCEMENT SHOWN IS TYPICAL UNLESS ADDITIONAL OR LARGER BARS ARE REQUIRED BY SEISMIC DESIGN.
3. IN LIEU OF 1.7 L_d LAP, BARS MAY BE WELDED. SEE TABLE BELOW.



TYPICAL WALL CORNER & JAMB — TYPICAL WALL INTERSECTION

NOTE: FOR ℓ_d, DEVELOPMENT LENGTH IN TENSION,

TABLE 6-2

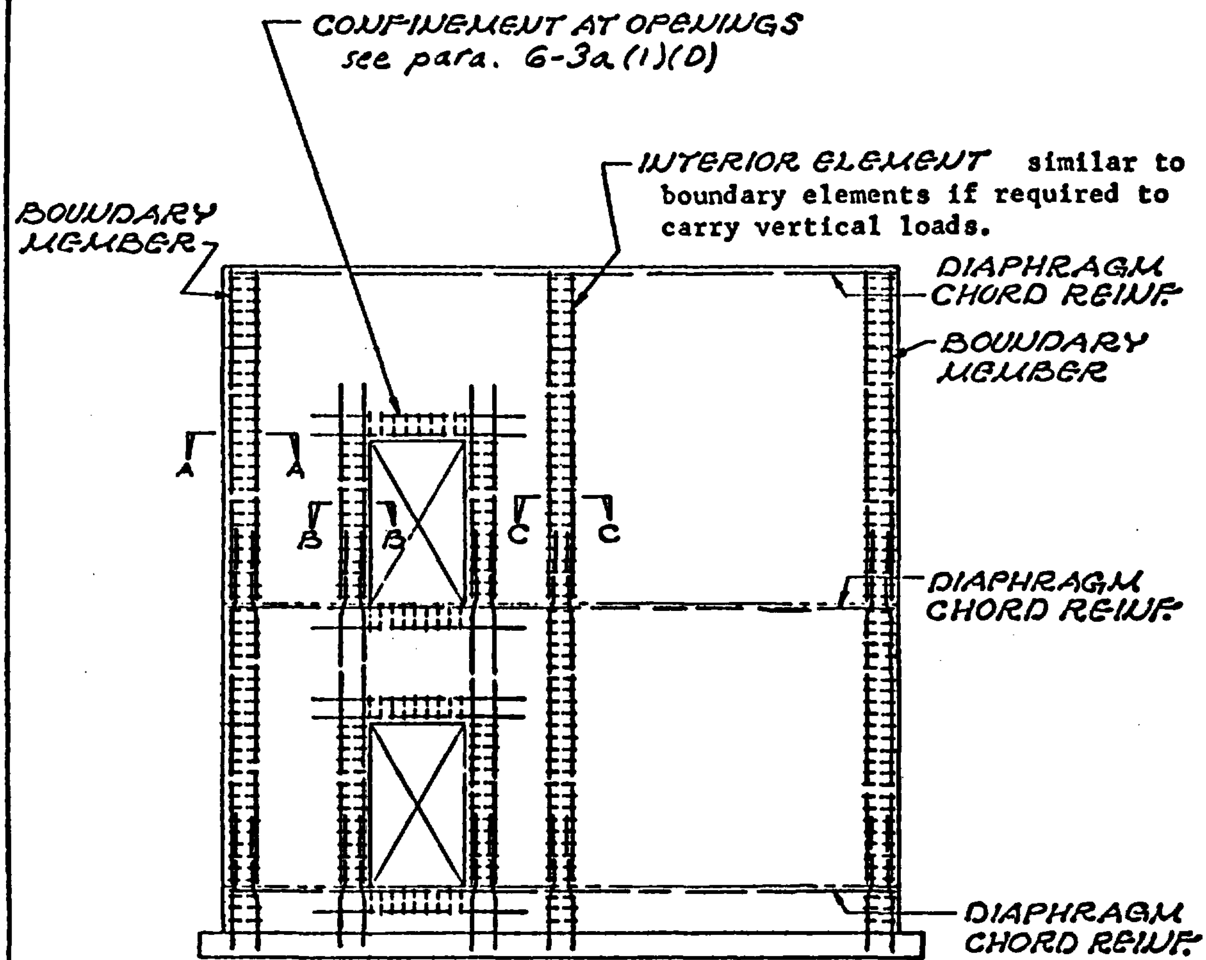
MINIMUM CONCRETE WALL REINF.	
WALL THICKNESS	VERTICAL & HORIZONTAL REINFORCING O
5"	#4 @ 16" o.c. IN CENTER
6"	#4 @ 13" o.c. IN CENTER
7"	#4 @ 11" o.c. IN CENTER
8"	#4 @ 10" o.c. IN CENTER

O SPACING OF BARS NOT TO EXCEED d/3 WHERE d IS DIMENSION OF THE WALL PIER PARALLEL TO SHEAR FORCE

TABLE 6-3

MINIMUM LENGTH OF STANDARD AWS. FLARE GROOVE WELDS TO DEVELOP LAPPED REINFORCING BARS	
BAR	WELD LENGTH (EACH SIDE)
3	2"
4	2½"
5	3"
6	3½"
7	4"

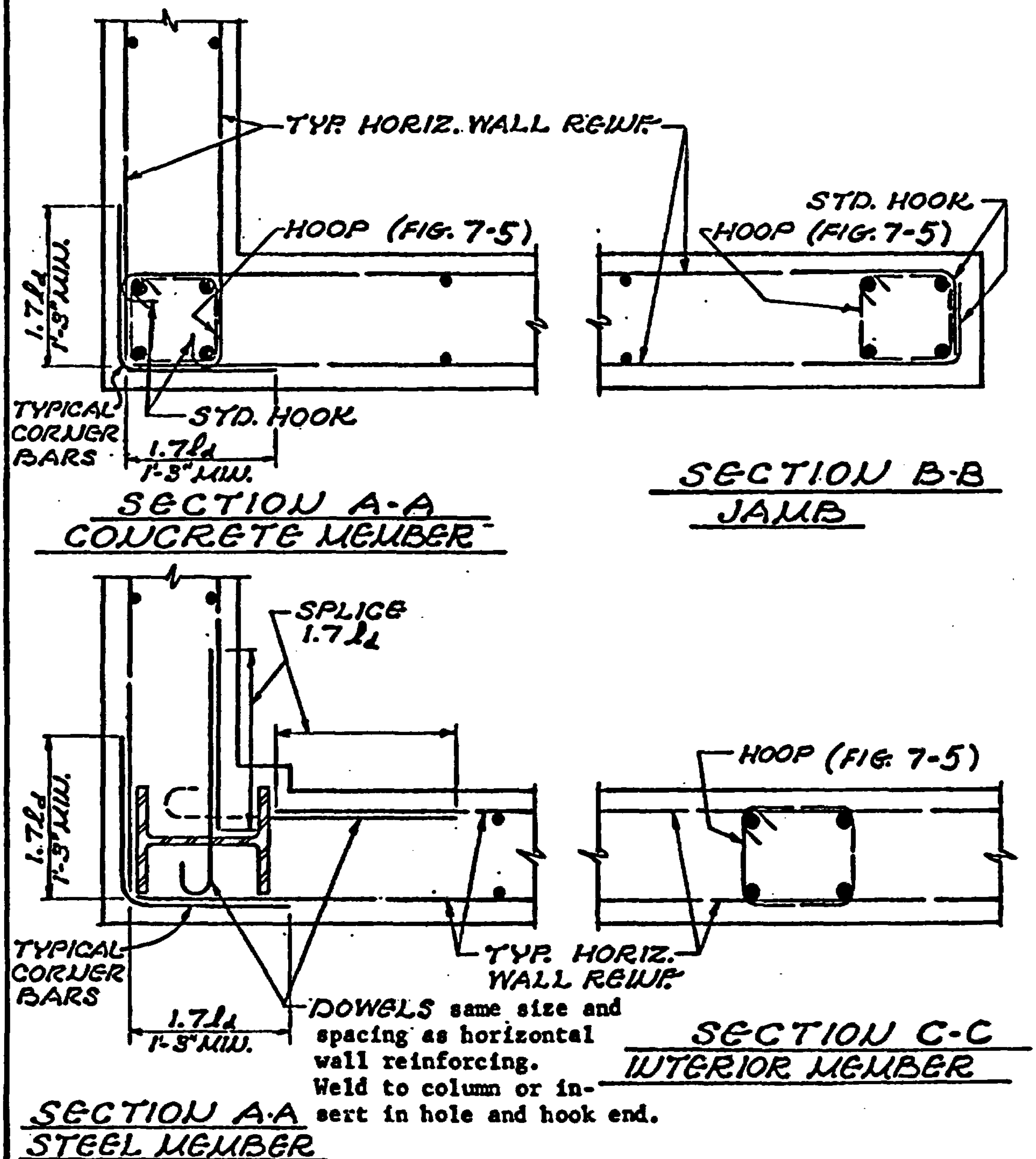
Figure 6-8B Minimum Concrete Shear Wall Reinforcement (One curtain)



WALL ELEVATION WITH SPECIAL BOUNDARY ELEMENTS

NOTES: For Sections A-A, B-B, and C-C, see Figure 6-10.
Special vertical boundary members, as shown above, shall be provided at the edges of concrete shear walls designated as Shear Wall Type A (paragraph 6-3a(1)(D)).

Figure 6-9. Shear Wall Type A - Special Boundary Members



NOTES: For location of sections, see Figure 6-9.
For l_d, development length in tension, see ACI318-77, Sect. 12.2.
Steel boundary members shall conform with paragraph 6-3a(1)(D).

Figure 6-10. Sections Through Special Boundary Elements

(2) *Classification of concrete shear walls and concrete braced frames.* Concrete shear walls and braced frames are classified under three categories for use in table 3-7 in section 3-6.

(a) Shear Wall Type A. Reinforced concrete shear walls with vertical boundary members, designed in accordance with the provisions of paragraph 6-3*a*(1), are classified as Shear Wall Type A.

(b) Shear Wall Type B. Reinforced concrete shear walls designed similar to Shear Wall Type A, with the exception of paragraph 6-3*a*(1)(D) (i.e., special vertical boundary elements are not required), are classified as Shear Wall Type B.

(c) Braced frames. Reinforced concrete braced frames will be designed in accordance with the provisions of paragraph 6-3*a*(1)(B).

b. Discussion of Wall Deflections, Shear Distribution, and Assumptions

(1) *Wall deflections.* The deflection of a concrete shear wall is the sum of the shear and flexural deflections. In the case of a solid wall with no openings the computations of deflection are quite simple. However, where the shear wall has openings in it, as for doors and windows, the computations for deflection and rigidity are much more complex. An exact analysis, considering angular rotation of elements, rib shortening, etc., is very time consuming. For this reason, several short-cut approximate methods involving more or less valid assumptions have been developed. These do not always give consistent or satisfactory results. Therefore, conservative approach and judgment must be used. Refer to paragraph 6-2*d*(2) for additional discussion.

(2) *Shear distribution.* It is necessary to make a logical and consistent distribution of story shears to each wall. Rigidity analysis is discussed in chapter 4, paragraph 4-4*e*, and in paragraph 6-2*d* of this chapter. An exact determination of the story shear distribution is very difficult and is not necessary. Approximate methods in which the deflections of portions of walls are combined usually are adequate. Examples illustrating various methods of rigidity computations are shown in appendix C.

(3) *Deflection charts.* Deflection charts for fixed-ended corner and rectangular piers are shown in figure 6-11. Curves 5 and 6 are for cantilever corner and rectangular piers. The corner pier curves are for the special case where the I (moment of inertia) of the corner pier is 1.5 times the I of a rectangular pier. For other I values the bending portion of the deflection would be proportional. The deflections shown on the charts are for a horizontal load P of 1,000,000 pounds. The deflections shown on the charts are reasonably accurate. The formulas written on the curves can be used to check the resul' However, the charts will give no better results tl the assumptions made in the shear wall analysis. For instance, the point of contraflexure of a vertical pier may not be in the center of the pier height. In some cases the point of contraflexure may be selected by judgment and an interpolation made between the cantilever and fixed conditions.

(4) *Assumptions*

(a) The foundation is unyielding or that soil pressures will vary as a straight line under a wall when subjected to overturning. These may not always be realistic assumptions, but are generally adequate for design purposes.

(b) Where the openings in a shear wall are so large that the resulting wall approaches an assembly similar to a rigid frame (h/d values off the chart), the wall will be analyzed as a rigid frame.

c. Construction Joints and Dowels. The contact faces of shear wall construction joints have exhibited slippage and related drift damage in past earthquakes. Consideration must be given to location and details of construction joints. They must be clean and roughened. It is highly desirable to provide intermittent shear keys in Seismic Zone Nos. 3 and 4. Shear friction reinforcement may be provided in accordance with ACI (318-77) Section 11.7. A c efficient of friction of 0.6 is suggested to account seismic effects.

6-4. Tilt-up and other precast concrete shear walls. *a. Analysis.* Where tilt-up or precast concrete walls are used as shear walls, the basic analysis is the same as that for walls of cast-in-place concrete. In this case the boundary conditions become critical and the shears between precast and cast-in-place elements must be analyzed. Shears between two precast elements or between a precast element and a cast-in-place element may be developed by shear keys, dowels, or welded inserts. The contact joint itself is a cold joint and will be given no shear or tension value.

b. Joints. Weakened plane joints are frequently provided in poured-in-place concrete to route cracks caused by shrinkage or temperature change. These joints normally do not affect the analysis of shear walls. However, in precast concrete elements, joints are frequently provided which structurally separate one element from another. In the case of precast wall construction, for instance, one might have a series of concrete elements tied together at top and bottom but structurally separated from each oth by vertical joints. Since all elements in a line are t together at the top they must have equal horizon

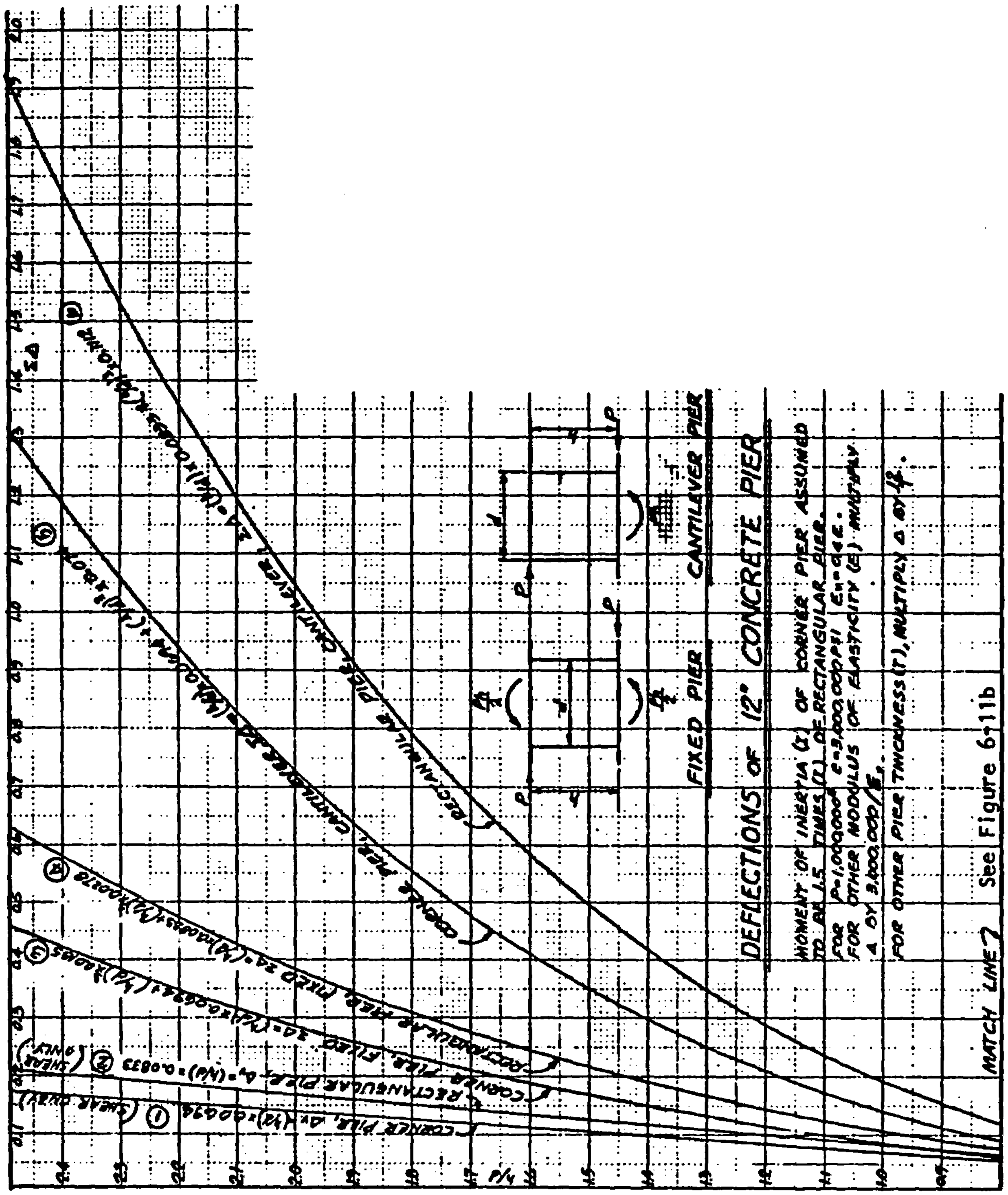


Figure 6-11a. Design Curves for Masonry and Concrete Shear Walls

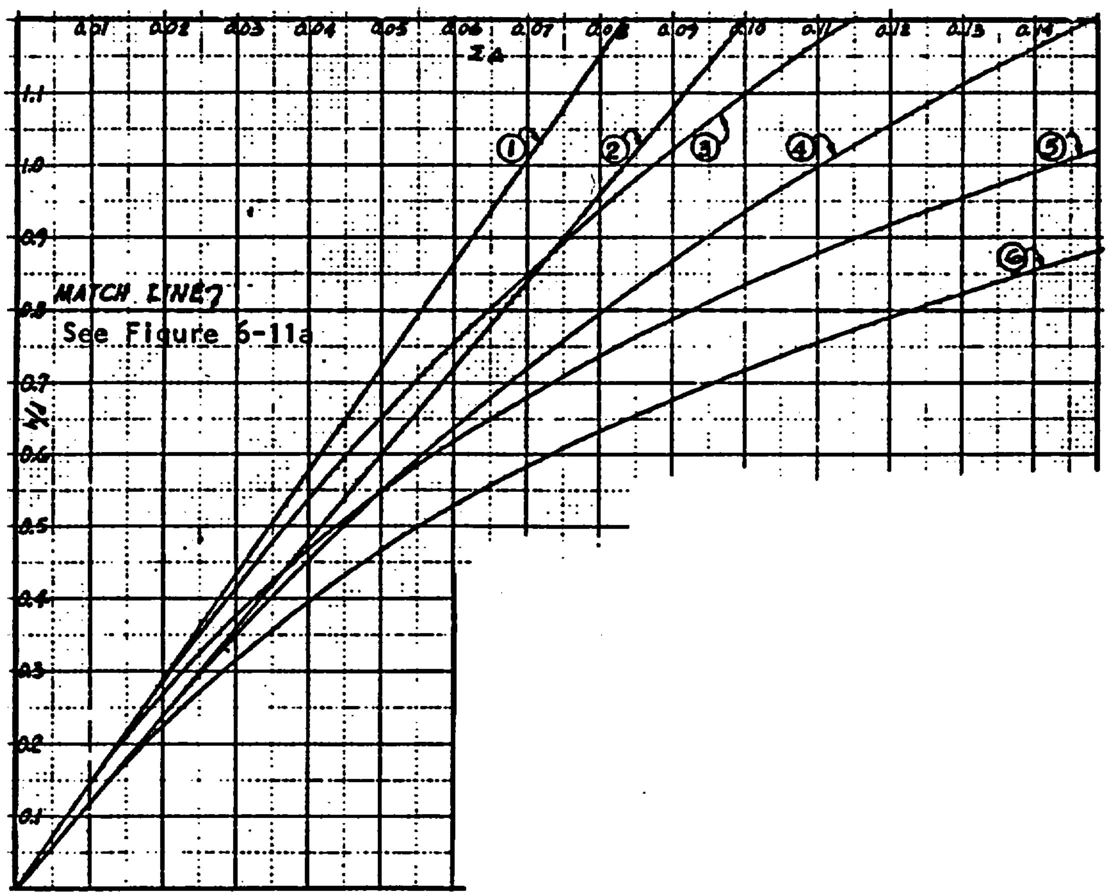


Figure 6-11b. Design Curves for Masonry and Concrete Shear Walls

deflections and therefore a horizontal force parallel with the line of units will be resisted by the individual elements in proportion to relative rigidities. Such elements may not have equal rigidities since some may contain large openings or may be of different height-width ratios. Some elements may deflect primarily in shear and others primarily in flexure. Where significant dissimilar deflections are found, the building elements tying the individual units together must be analyzed to determine their ability to resist or accept such deformations including angular rotation without losing their ability to function as ties or diaphragm chords or footings. The use of mechanical keys or sleeved dowels may be used to assist in eliminating differential movement of adjacent precast panels separated by control joints where appearance and weather-tightness are otherwise satisfactorily provided.

c. Connectors for Shear Walls. Past experience indicates that the performance of weld plates or other nonductile connectors has been poor and in many cases they have resulted in failures during earthquakes. These connectors have been weak links in the shear wall connection. It is important that the load bearing shear walls be more stringently or conservatively designed since any connector failure during an earthquake may result in progressive failure to collapse. Therefore, all connectors for load and nonload bearing walls will be designed for three times the actual seismic shear forces. The shear force will be uniformly distributed throughout the height or length of the shear wall with reasonably spaced connectors (maximum spacing 4'-0") rather than with a few which will have localized concentration of stresses. Detailed calculations will be made including the localized effects in concrete walls attributed from these connectors. Sufficient details of connectors and embedded anchorage will be provided to preclude construction deficiency.

d. Typical Details. Refer to figure 6-12 for typical details of attachments.

6-5. Wood stud shear walls. *a. Working Shears Except Plywood.* Figure 6-13 gives in tabular form the maximum height-width ratios and allowable shear per lineal foot for wood stud shear walls with various types of sheathing or plaster except for plywood sheathed walls. The usual 33-1/3 percent increase for short-time seismic loads is *not* applicable to these allowable shear values. The strength of any wood stud shear wall may be made up of a combination of the materials listed. In no case shall the allowable shears for combinations of materials exceed 600 pounds per lineal foot.

b. Working Shears for Plywood. Details of plywood sheathed walls are shown on figure 6-14 and the allowable working shears are shown in figure 6-15. When a combination of plywood and other materials is used, the shear strength of the walls will be determined by the values permitted for plywood alone (fig 6-15).

c. Deflections. The deflection of wood frame shear walls at the present time is not readily computable. The maximum height-width limitations given herein are presumed to satisfactorily control deflections. Relative stiffnesses of wood stud shear walls will be measured by the effective lineal width of walls or piers between openings.

d. Let-In Brace. Except when used in combination with diagonal sheathing or plywood, a one-inch by four-inch brace let into the studs may be used to resist an additional horizontal force not exceeding 1,000 pounds, provided the total value of the shear wall does not exceed 600 pounds per foot. Each such brace shall be nailed to each stud and to the top and bottom plates with two 8d nails.

e. Wall Tie-Down. The end studs of any plywood sheathed shear wall and/or shear wall pier will be tied down in such a manner as to resist the overturning forces produced by seismic forces parallel to the shear wall. This overturning force is sometimes of sufficient magnitude to require special steel attachment details. A commonly used detail is shown on figure 6-16. Tie-downs will be computed using the required stresses for wood and its fastenings increased 33-1/3 percent for seismic forces.

6-6. Steel stud walls. Some small structures may be constructed using steel stud structural walls. In order for this type of wall to be capable of acting as a shear wall, some form of bracing is required. When the design forces permit, the detail shown on figure 6-17a may be used to resist a total of 1,000 pounds. In larger buildings where the design forces become greater, this method is impractical and other shear wall systems may be required. Figure 6-17b shows typical details at top of walls.

6-7. Vertically braced frames. *a. General Design Criteria.* The criteria governing the design of vertical braced frames will be chapter 3, paragraph 3-3(J)1g, paragraph 6-2 of this chapter, and as prescribed in this paragraph.

(1) *Structural steel braced frames.* Members of braced frames will be composed of ASTM A36, A441, A500 (Grades B and C), A501, A572 (Grades 42, 45, 50, and 55), or A588 structural steel and will conform to the AISC "Specification for Design,

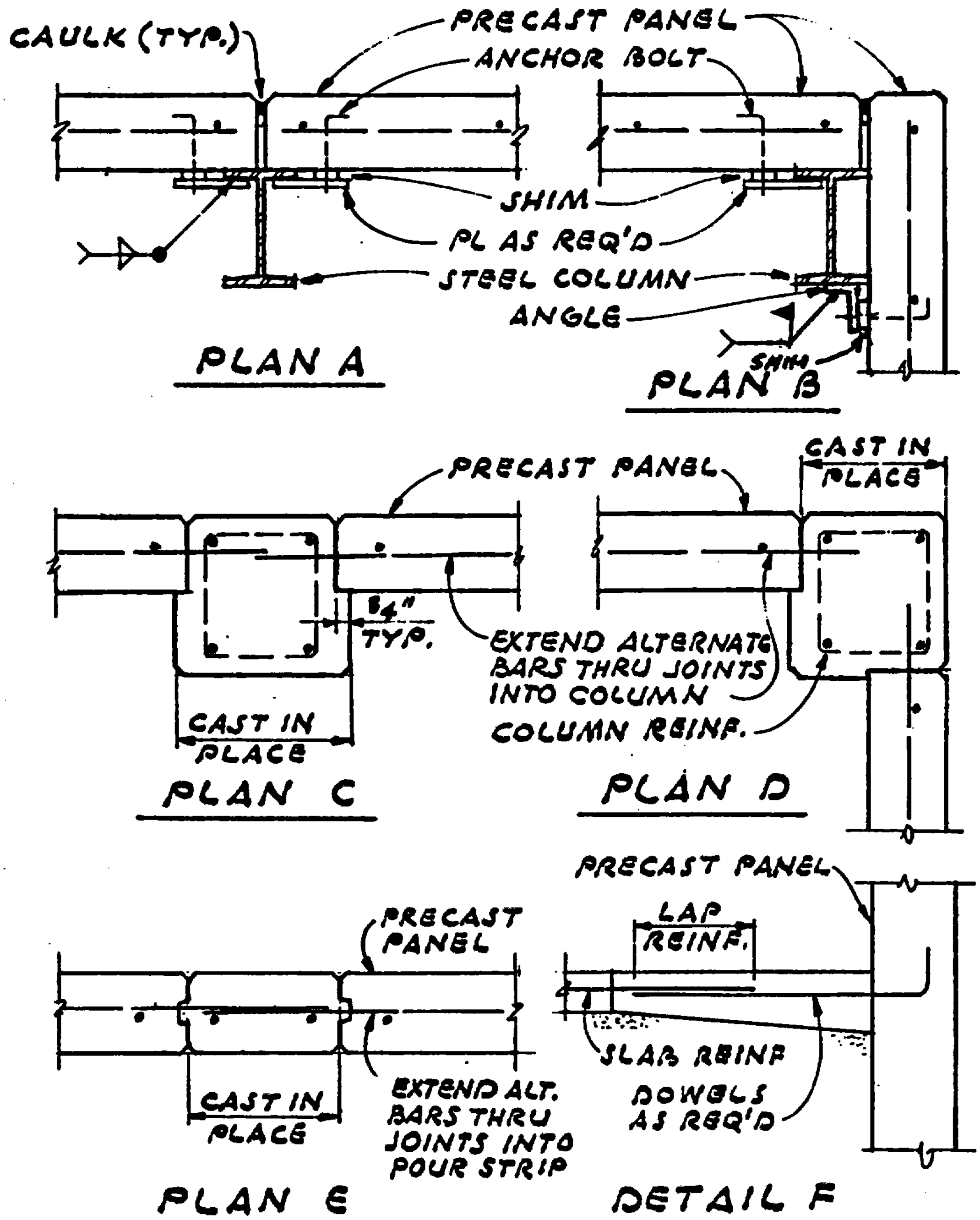
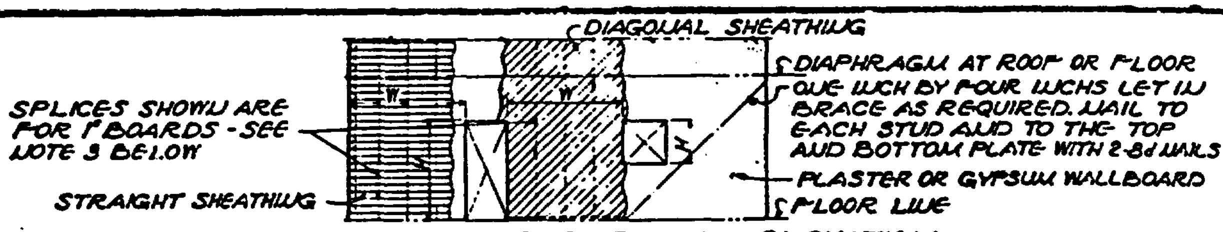


Figure 6-12. Tilt-up and Other Precast Concrete Walls Typical Details of Attachments



TYPICAL SHEAR WALL ELEVATION

VERTICAL SHEAR WALLS ONE SIDE ONLY	NAILING @ EACH BEARING COMMON UNLESS NOTED	MAXIMUM PIER HEIGHT-WIDTH RATIOS (H/W)	ALLOWABLE SHEAR LBS/LIN. FT.
1" STRAIGHT SHEATHING	2-8d	2:1	50
2" STRAIGHT SHEATHING	3-16d		40
CONVENTIONAL 1" DIAGONAL SHEATHING 1"x6"	2-8d(3-8d AT BOUNDARIES)	2:1	300
CONVENTIONAL 1" DIAGONAL SHEATHING 1"x8"	3-8d(4-8d AT BOUNDARIES)		
SPECIAL DIAGONAL SHEATHING	3-16d	3½:1	600
LATH & GYPSUM PLASTER	1¼" NO. 13 GAUGE 19/64" DIAMETER HEAD BLUED NAIL @ 5" o.c.	2:1	100
METAL LATH & PORTLAND CEMENT PLASTER	4d BLUED BOX NAILS @ 6" o.c. OR 1½" NO. 11 GAUGE 7/16" DIA. HEAD BARBED NAILS @ 6" o.c.	2:1	200
GYPSUM LATH, PLAIN OR PERFORATED 3/8" WITHOUT BLOCKING & 1/2" PLASTER	1⅛" NO. 13 GAUGE 19/64" DIAMETER HEAD BLUED NAIL @ 5" o.c.	1½:1	100
GYPSUM SHEATHING BOARD 1/2"x2'x8' WITHOUT BLOCKING	1¾" NO. 11 GAUGE 7/16" DIAMETER HEAD DIAMOND POINT. GALVANIZED @ 4" o.c.	1½:1	75
GYPSUM SHEATHING BOARD 1/2"x4' WITH BLOCKING	1¾" NO. 11 GAUGE 7/16" DIAMETER HEAD DIAMOND POINT GALVANIZED @ 4" o.c.	1½:1	175
GYPSUM WALLBOARD (DRYWALL) 1/2" WITHOUT BLOCKING	5d OR @ 7" o.c.	1½:1	100
	1¼"x.098 GA. @ 4" o.c.	1½:1	125
GYPSUM WALLBOARD (DRYWALL) 1/2" WITH BLOCKING	5d OR @ 7" o.c.	1½:1	125
	1¼"x.098 GA. @ 4" o.c.	1½:1	150
GYPSUM WALLBOARD (DRYWALL) 5/8" WITH BLOCKING	6d @ 4" o.c.	1½:1	175
GYPSUM WALLBOARD (DRYWALL) 5/8" TWO-PLY WITH BLOCKING	6d @ 9" o.c. BASE PLY 8d @ 7" o.c. FACE PLY	1½:1	250

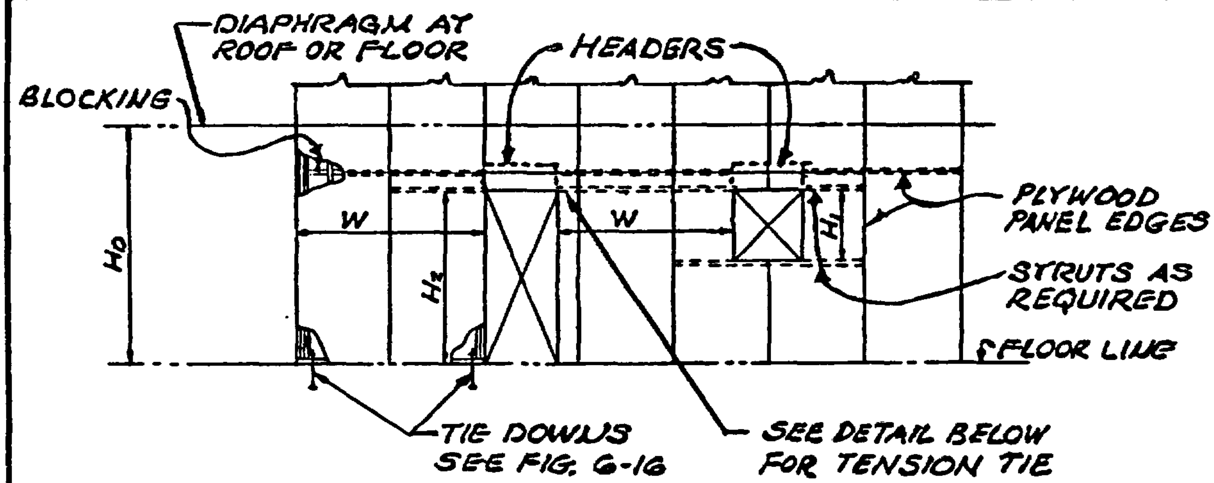
NOTES:

1. VALUES SHALL BE MODIFIED FOR PARTICULAR SPECIES OF WOOD IN ACCORDANCE WITH PERCENTAGES BELOW

SPECIES	% OF TABULATED VALUES	SPECIES	% OF TABULATED VALUES	SPECIES	% OF TABULATED VALUES
DOUGLAS FIR (WEST COAST & INLAND)	100%	SPRUCE (SITKA)	75%	PINE (PONDEROSA)	65%
LARCH	100%	FIR (WHITE)	70%	PINE (SUGAR)	65%
HEMLOCK (WESTERN)	85%	CEDAR (WESTERN RED)	65%	PINE (LODGEPOLE)	65%
REDWOOD	80%	PINE (IDAHO WHITE)	65%	SPRUCE (ENGLEMANN)	65%

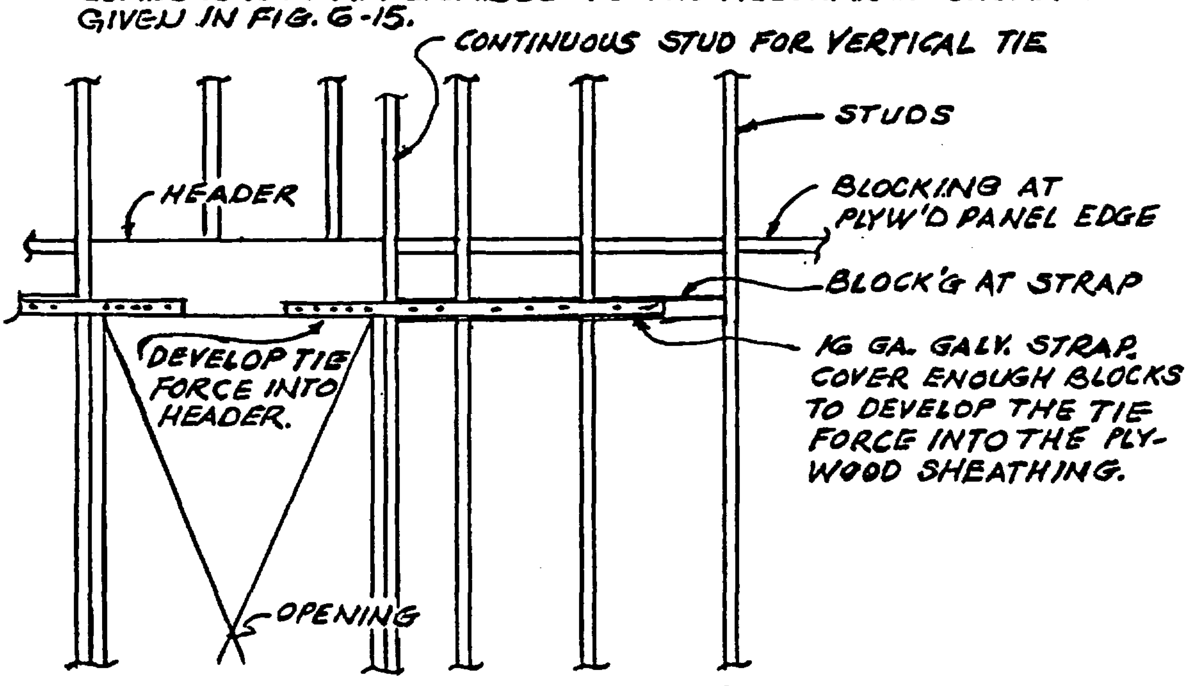
2. IF USED UNDER CONDITIONS OTHER THAN CONTINUOUSLY DRY, VALUES FOR WOOD SHEAR WALLS SHALL BE REDUCED TO 67% OF THE TABULATED VALUES.
3. DIAGONAL OR STRAIGHT SHEATHING - END JOINTS OF ADJACENT BOARDS WILL BE SEPARATED BY AT LEAST TWO JOIST OR RAFTER SPACES WITH AT LEAST TWO BOARDS BETWEEN JOINTS ON SAME SUPPORT.
4. SPECIAL DIAGONAL SHEATHING SHALL CONSIST OF TWO LAYERS OF 1" CONVENTIONAL DIAGONAL SHEATHING AT 90° TO EACH OTHER AND ON THE SAME FACE OF STUDS.
5. TYPE OF NAILS, SEE APPLICABLE AGENCY GUIDE SPECIFICATIONS.
6. THESE SHEAR WALLS SHALL NOT BE USED TO RESIST FORCES DUE TO CONCRETE NOR MASONRY MASSES, EXCEPT FOR VENEERS, SHOWER STALLS AND MINOR CONCRETE FILLS SUCH AS EQUIPMENT PADS.
7. THE USUAL 33⅓% INCREASE FOR SHORT-TIME SEISMIC LOADS IS NOT APPLICABLE TO THESE ALLOWABLE SHEAR VALUES.
8. SHEAR WALLS USING A COMBINATION OF MATERIALS WILL BE CONSIDERED AS THE SUM OF VALUES OF THE INDIVIDUAL MATERIALS NOT TO EXCEED A MAXIMUM OF 600 POUNDS PER LINEAL FOOT.
9. IF USED IN COMBINATION WITH ANY OF THE SHEAR WALLS LISTED, A LET IN BRACE MAY BE USED TO RESIST AN ADDITIONAL FORCE OF 1000 POUNDS, WITH TOTAL NOT TO EXCEED 600 POUNDS PER LINEAL FOOT

Figure 6-13. Typical Wood Stud Shear Walls of Various Materials Other Than Plywood



NOTES:

1. FOR VALUES OF PLYWOOD SHEATHED SHEAR WALLS, SEE FIG. 6-15.
2. HEIGHT-WIDTH RATIO OF PLYWOOD SHEATHED SHEAR WALLS WILL BE LIMITED TO 3½ TO 1. H_0/W WILL BE USED FOR THE HEIGHT-WIDTH RATIO UNLESS STRUTS ARE DEVELOPED AT THE TOP AND BOTTOM OF THE OPENINGS IN WHICH CASE H_1/W OR H_2/W MAY BE USED.
3. THESE SHEAR WALLS SHALL NOT BE USED TO RESIST FORCES DUE TO CONCRETE NOR MASONRY MASSES, EXCEPT FOR VENEERS, SHOWER STALLS AND MINOR CONCRETE FILLS SUCH AS EQUIPMENT BASES.
4. THE USUAL 33⅓% INCREASE FOR SHORT-TIME SEISMIC LOADS IS NOT APPLICABLE TO THE ALLOWABLE SHEAR VALUES GIVEN IN FIG. 6-15.



DETAIL OF TENSION TIE - STRUT

Figure 6-14. Plywood Sheathed Wood Stud Shear Walls

Recommended Shear in Pounds Per Foot for Plywood Shear Walls with Framing of Douglas Fir, Larch, or Southern Pine (a) For Wind or Seismic Loading (b)

Plywood Grade	Minimum Nominal Plywood Thickness (in.)	Minimum Nail Penetration in Framing (in.)	Plywood Applied Direct to Framing					Plywood Applied Over 1/2" Gypsum Sheathing				
			Nail Size (common or galvanized box)	Nail Spacing at Plywood Panel Edges (in.)				Nail Size (common or galvanized box)	Nail Spacing at Plywood Panel Edges (in.)			
				6	4	2½	2		6	4	2½	2
STRUCTURAL I C-D INT-APA, or STRUCTURAL I C-C EXT-APA	5/16	1-1/4	6d	200	300	450	510	8d	200	300	450	510
	3/8	1-1/2	8d	230(d)	360(d)	530(d)	610(d)	10d	280	430	640(e)	730(e)
	1/2	1-5/8	10d	340	510	770(e)	870(e)	—	—	—	—	—
C-D INT-APA C-C EXT-APA STRUCTURAL II C-D INT-APA STRUCTURAL II C-C EXT-APA APA panel siding (f) and other APA grades except species Group 5.	5/16 or 1/4 (c)	1-1/4	6d	180	270	400	450	8d	180	270	400	450
	3/8	1-1/2	8d	220(d)	320(d)	470(d)	530(d)	10d	260	380	570(e)	640(e)
	1/2	1-5/8	10d	310	460	690(e)	770(e)	—	—	—	—	—
APA panel siding (f) and other APA grades except species Group 5.	5/16 (c)	1-1/4	Nail Size (galvanized casing) 6d	140	210	320	360	Nail Size (galvanized casing) 8d	140	210	320	360
	3/8	1-1/2	8d	130(d)	200(d)	300(d)	340(d)	10d	160	240	360	410

(a) For framing of other species: (1) Find species group of lumber in the NFPA Nat'l Design Spec. (2) (a) For common or galvanized box nails, find shear value from table for nail size, and for STRUCTURAL I plywood (regardless of actual grade). (b) For galvanized casing nails, take shear value directly from table. (3) Multiply this value by 0.82 for Lumber Group III or 0.65 for Lumber Group IV.

(b) All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails 6 inches o.c. along intermediate members for 3/8-inch plywood with face grain parallel to studs spaced 24 inches o.c. For other conditions and plywood thicknesses, space nails 12 inch o.c. on intermediate supports.

(c) 3/8-inch or 303-16 o.c. is minimum recommended when applied direct to framing as exterior siding.

(d) Shears may be increased 20 percent provided (1) studs are spaced a maximum of 16 inches o.c., or (2) plywood is applied with face grain across studs, or (3) plywood is 1/2-inch or greater in thickness.

(e) Reduce tabulated shears 10 percent when boundary members provide less than 3-inch nominal nailing surface.

(f) 303-16 o.c. plywood may be 5/16-inch, 3/8-inch or thicker. Thickness at point of nailing on panel edges governs shear values.

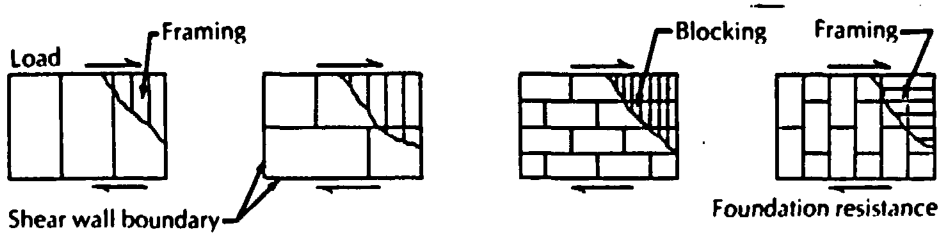
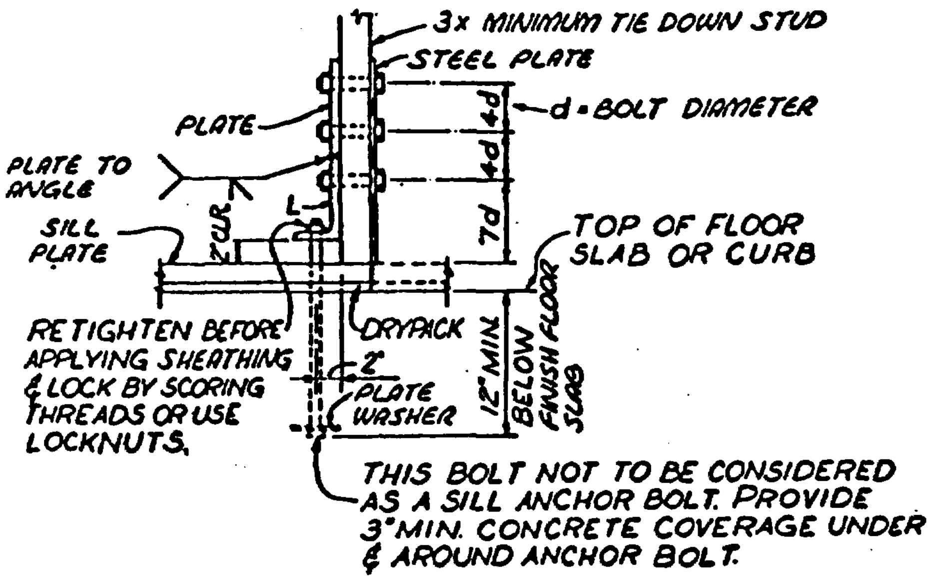
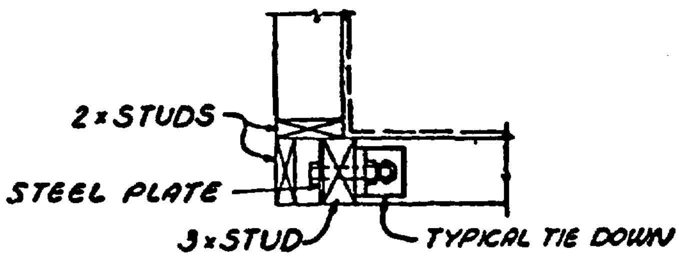


Figure 6-15. Working Stresses for Plywood Sheathed Wood Stud Walls



TYPICAL TIE-DOWN DETAIL A



PLAN B
CORNER DETAIL AT TIE-DOWN

NOTE: Angle, bolts, plates, posts, footings, etc., to be designed for uplift.

Figure 6-16. Wood Stud Walls - Typical Tie-Down Details

Fabrication, and Erection of Structural Steel Buildings."

(2) *Reinforced concrete braced frames.* Will conform to the requirements of paragraph 6-3*a*(1)(B).

(3) *Wood braced frames.* Wood braced frames will be designed using normal procedures illustrated in many easily obtainable texts and are not covered in this manual. "National Design Specifications for Wood Construction" (1977 Edition and 1980 Supplement), NFPA, applies.

b. General Discussion.

(1) *Definition of braced frame.* In chapter 3, paragraph 3-3(B), a braced frame is defined as a truss system or its equivalent which is provided to resist lateral forces and in which the members are subjected to axial stresses. The determination of whether a bracing system, such as one utilizing deep knee braces, is a braced frame or a moment resisting frame is explained in the 1960 SEAOC Commentary (p. 32) as follows: "If the deflection of a braced bent is predominantly due to bending and rotation of individual members rather than the direct stress distortion of shear carrying bracing members, it may be considered a frame; if it deflects primarily due to the distortion of the shear carrying member it is a shear wall." Braced frames may be made of any approved structural material (para 6-2*e*). Braced frames may be of various forms. The X-braced panels, consisting of diagonal tension members and vertical compression members, are most frequently used (fig 6-18). Trussed portal bracing or K-bracing is frequently used to permit unobstructed openings (fig 6-20). Braced frames with single diagonal members capable of taking compression as well as tension are used to permit flexibility in the location of openings (fig 6-19). The deflection of braced frames is readily computed using recognized methods.

(2) *Function of braced frame.* The function of the bracing is that of resisting forces that tend to deform the building in a direction parallel to the plane of that bracing, and to transmit these lateral loads to the foundation. As with other systems, the deformations to be expected in a major earthquake can be much greater than those found using the prescribed forces. As the ductility of the usual braced systems has not been adequately demonstrated, multiple braces should be used whenever possible (see para 6-2*e*).

(3) *Connections.* Obviously, a member will not support loads in excess of what its connections and other details can hold. As a general principle, these details should be sufficient to develop the useful strength of the member or part concerned, regardless of calculated stress. In lieu of developing the full capacity of the member or part concerned, the connections will be designed for 1.25 times the design force without the one-third increase usually permitted.

c. Special Requirements for Braced Frames. Refer to chapter 3, paragraph 3-3(J)1g, for special load factor and connection requirements for braced frames. Reference should also be made to the SEAOC Commentary, pages 47-C and 48-C.

6-8. Masonry shear walls. Distribution of shears to masonry walls will be in a similar manner as described for cast-in-place concrete walls. For typical masonry shear wall details, see chapter 8, Reinforced Masonry. When masonry shear walls are used as part of a dual system (i.e., K=0.8 per category 3 in table 3-3) in Seismic Zones 2, 3, or 4, special vertical boundary elements are required. These elements will be composed of structural steel or reinforced concrete in accordance with paragraph 6-3*a*(1)(D) or will be composed of masonry columns or pilasters in accordance with chapter 8, paragraph 8-14.

6-9. Metal wall systems. Metal wall panels or sidings less than 22 gage are not permitted for use as shear walls. Metal decking and attachments complying with chapter 5, paragraph 5-6 will be permitted for use as shear wall diaphragms.

> EXCEPTION: In Seismic Zone 1, a pre-engineered metal building with panels less than 22 gage requires that load tests be submitted for evaluation and approval.

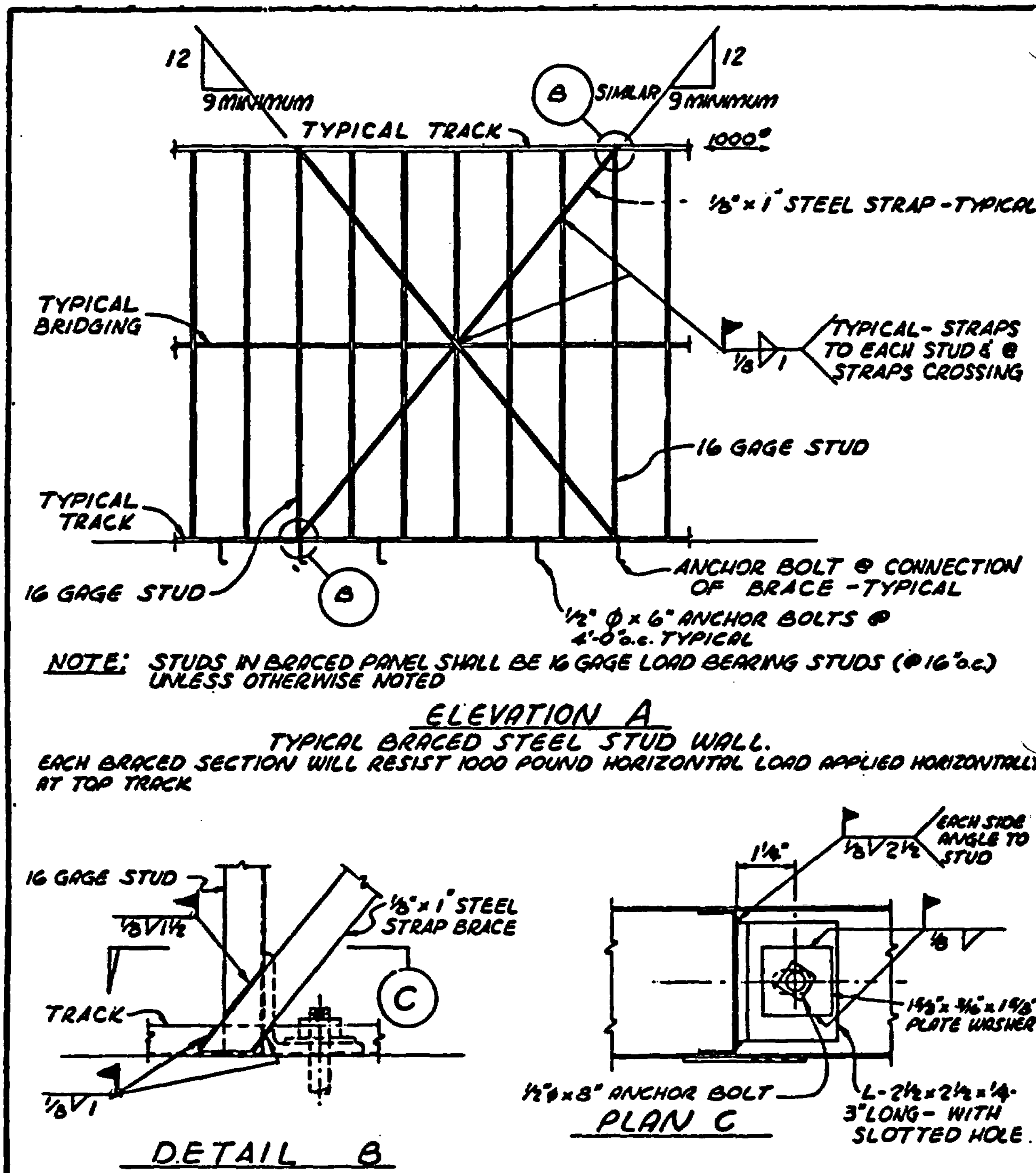


Figure 6-17a. Steel Stud Shear Walls Typical Details

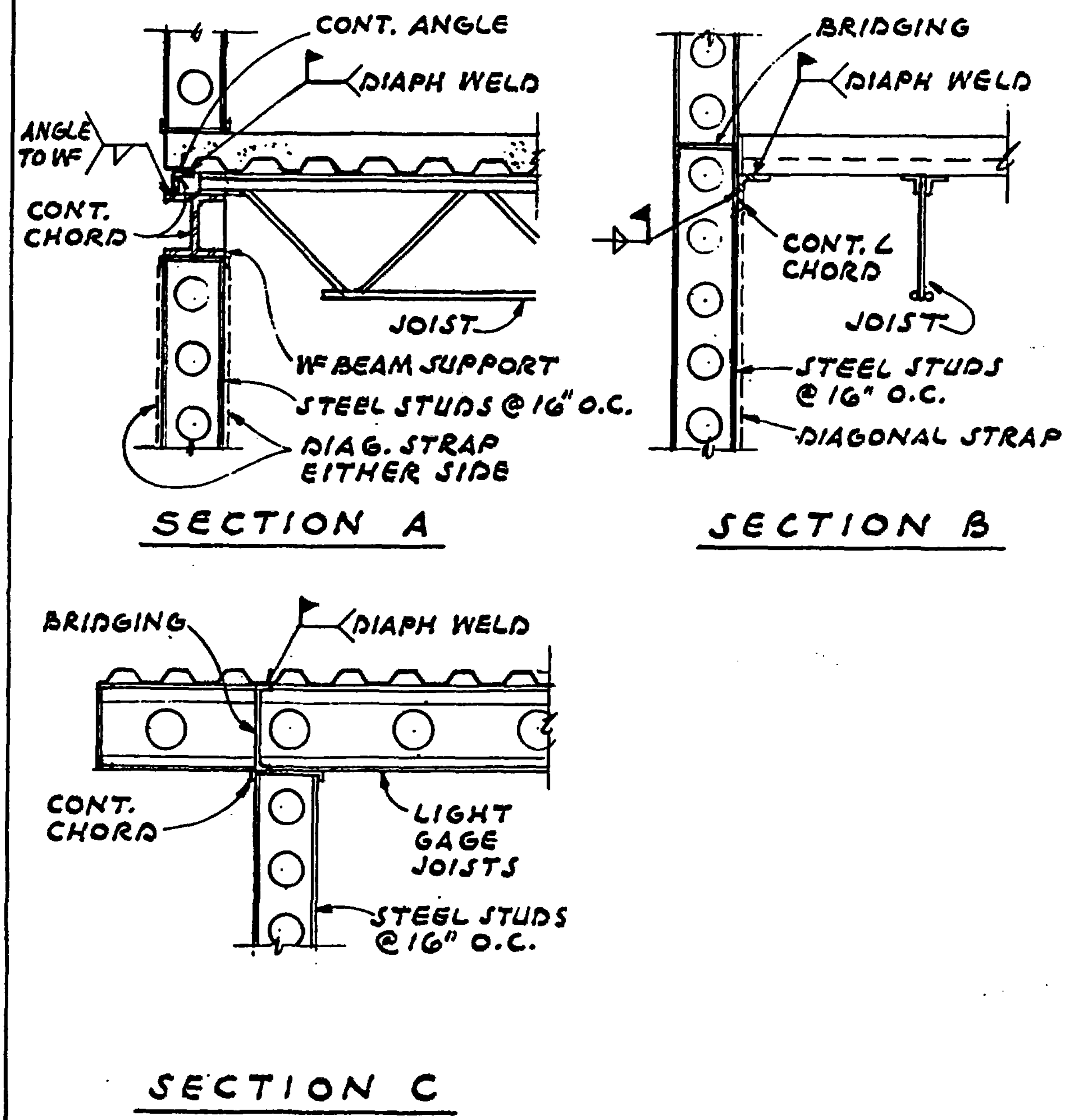


Figure 6-17b. Steel Stud Shear Walls Typical Details

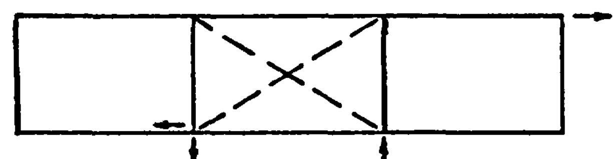


Fig. 6-18. X-BRACED FRAME (diagonals in tension; verticals in tension or compression).

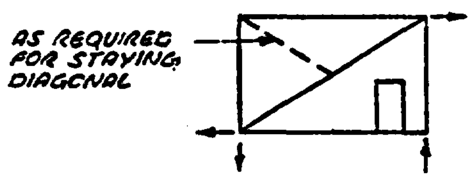


Fig. 6-19. BRACED FRAME (diagonals and verticals in compression or tension).

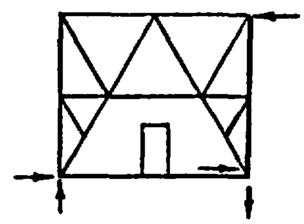


Fig. 6-20. PORTAL BRACED FRAME

CHAPTER 7
SPACE FRAMES

7-1. Purpose and scope. This chapter prescribes the criteria for design of moment resisting space frames of buildings in seismic areas; indicates principles, factors, and concepts involved in seismic design of moment resisting frames; gives design data; and illustrates typical details of construction. For braced frames which act as shear walls, refer to chapter 6.

7-2. General. A space frame, as defined in chapter 3, paragraph 3-3(B), 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.

a. Seismic Space Frames. Horizontal forces at any floor or roof level are transmitted to the foundation (ground) by using the strength, rigidity, and ductility of a moment resisting space frame. A seismic space frame will be based on the assumption that the frame depends on its own bending stiffness for the lateral stability of the structure (fig 7-1). It is important to remember that deformations resulting from the dynamic response of a major earthquake are much greater than those determined from the application of the prescribed forces. This means that a space frame that conforms to the minimum requirements of this manual will survive a major earthquake only if it can yield without essential loss of lateral resistance or vertical load capacity. Since normal building materials have very limited energy-absorbing capacity in the elastic range of action, it follows that what is needed is a large energy capacity in the inelastic range. The term "ductility" is used to denote this property. Providing a ductile seismic frame may well prove to be the difference between sustaining tolerable and, in many cases, repairable damage, instead of catastrophic failure. The energy dissipation, ductility, and structural response (deformation) of space frames depend upon the type of members, connections (joints), and materials of construction used. The behavior of joints is a critical factor in the efficiency of building frames during high intensity cyclic loading. A seismic space frame will be a moment resisting space frame or a ductile moment resisting space frame.

b. Moment Resisting Space Frames. A moment space frame is a vertical load-carrying space frame in which the members and joints are capable of resisting design lateral forces by bending moments. Although a moment resisting space frame need not comply with all the special requirements of a ductile moment resisting space frame, it will comply with the applicable requirements set forth in this chapter to qualify as a seismic space frame.

c. Ductile Moment Resisting Space Frames. To qualify for a K-factor of 0.67, the structural system for resisting lateral forces must be a *ductile* moment resisting space frame. A ductile moment resisting space frame will be required for any building of any height where a K-factor of less than 1 is used (some exceptions are permitted for dual systems as provided for in table 3-7). A *ductile* moment resisting space frame will be based on the assumption that the frame depends on its own bending stiffness for the lateral stability of the structure. Beams (or girders) shall be connected to columns by rigid joints which are capable of developing in the beams the full plastic capacity of the beams, under moment reversals. To take advantage of the energy absorbing capacity of the structural members, connections shall be designed to be at least as strong as the members connected. A ductile moment resisting space frame will be constructed of structural steel or reinforced concrete and will comply with the requirements of Concrete Frame Type A (para 7-3) or Steel Frame Type A (para 7-5). In Seismic Zone No. 1, Concrete Frame Type B (para 7-4*a*) qualifies as a ductile moment-resisting space frame.

d. Classification of Moment Resisting Space Frames. Space frames are classified under several categories in chapter 3, paragraph 3-6*a*, for use in table 3-7. The design criteria for Types A, B, and C of both concrete and steel moment resisting space frames are covered in paragraphs 7-3 through 7-6. Concrete Frame Type D, which is not classified as a moment resisting seismic space frame (although such a frame will naturally have some moment resistant capacity), is a vertical load-carrying space frame designed in accordance with ACI 318-77.

7-3. Concrete Ductile Moment Resisting Space Frame—Concrete Frame Type A.

a. General Design Criteria. The criteria used to design ductile moment resisting space frames will be ACI 318-77 except appendix A, and as modified by SEAOC Section 2 (reprinted below) and by this manual.

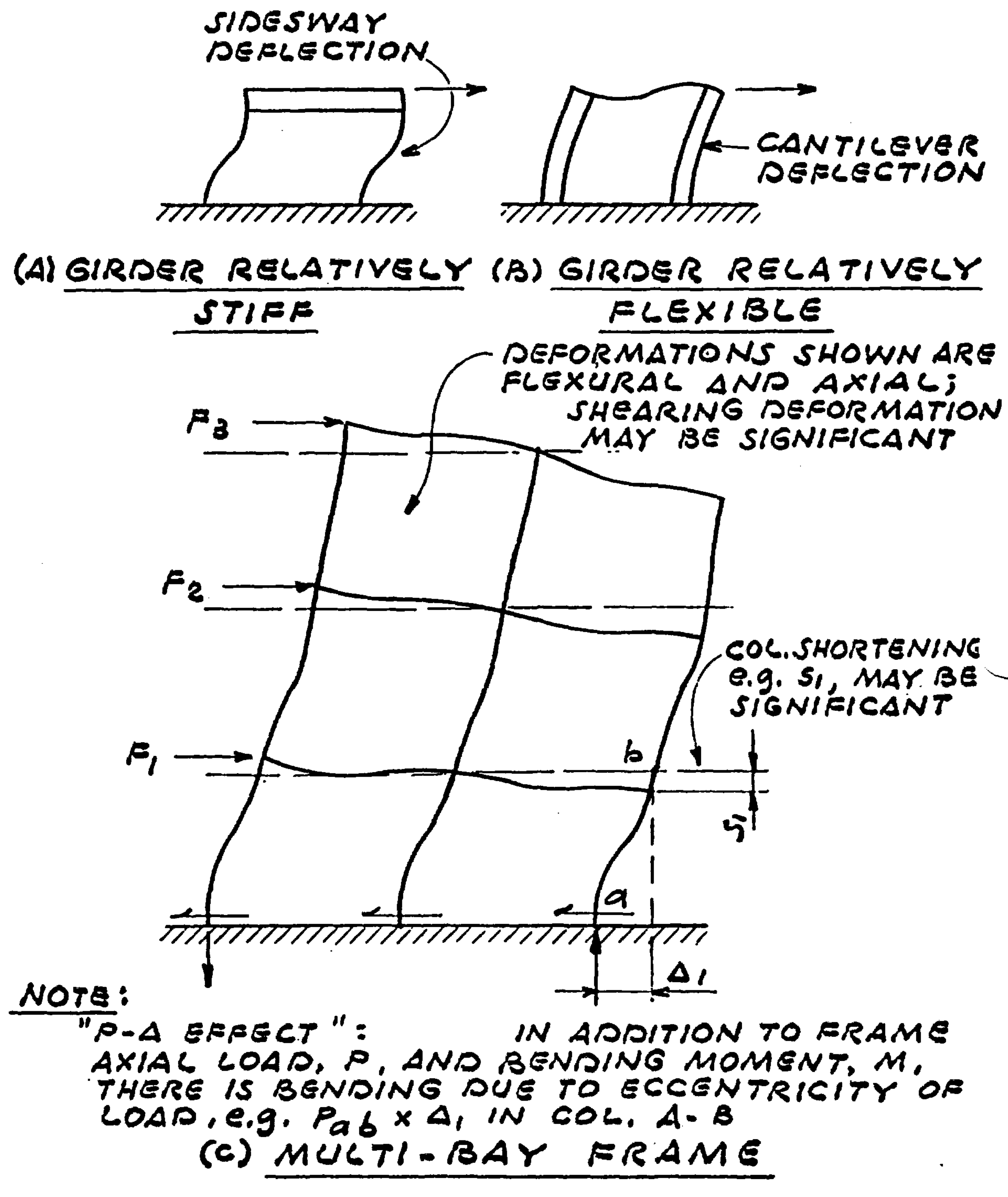


Figure 7-1 Frame deformations

(1) SEAOC Section 2, Concrete Ductile Moment Resisting Space Frames:[a] *(Modifications are in italics)*

(A) General.

Design and construction of cast-in-place, monolithic reinforced concrete framing members and their connections in ductile moment resisting space frames shall conform to the requirements of ACI Building Code, ACI 318, and all the requirements of *SEAOC* Section 2 *as modified herein.*

EXCEPTION: Precast concrete frame members may be used, if the resulting construction complies with all the provisions of this Section.

All lateral load resisting frame members shall be designed by the strength design method except that the alternate design method may be used provided that it is shown that the factor of safety is equivalent to that achieved with the strength design method.

ACI 318, for earthquake loading shall be modified to:

$$U=1.4(D+L)+1.4E \qquad (7\text{-}1)^{b}$$

$$U=0.9D+1.4E \qquad (7\text{-}2)$$

Members of space frames which are designed to resist seismic forces shall be designed, in accordance with the provisions of this Section, so that shear failures will not occur if the frame is subjected to lateral displacements in excess of yield displacements.

(B) Definitions.

CONFINED CONCRETE is concrete which is confined by closely spaced special transverse reinforcement restraining the concrete in directions perpendicular to the applied stresses.

SPECIAL TRANSVERSE REINFORCEMENT is composed of spirals, stirrup-ties, or hoops and supplementary cross-ties provided to restrain the concrete to make it qualify as confined concrete.

STIRRUP-TIES OR HOOPS are continuous reinforcing steel of not less than a No. 3 bar bent to form a closed hoop which encloses the longitudinal reinforcing and the ends of which have a standard 135 degree bend with a 10 bar diameter extension or equivalent.

(C) Physical Requirements for Concrete and Reinforcing Steel.

1. Concrete. The minimum specified 28-day strength of the concrete, f'_c, shall be 3000 pounds per square inch.

The maximum specified strength for lightweight concrete shall be limited to 4000 psi.

2. Reinforcement. All longitudinal reinforcing steel in columns and beams shall comply with ASTM A-615, grade 40 or 60. The actual yield stress, based on mill tests, shall not exceed the minimum specified yield stress, f_y, by more than 18,000 psi. Retests shall not exceed this value by more than an additional 3000 psi. In addition the ultimate tensile stress shall be not less than 1.33 times the actual yield stress, based on mill tests.[c] Grades other than these specified for design shall not be used.

[a]From the publication "Recommended Lateral Force Requirements and Commentary" by the Seismology Committee, Structural Engineers Association of California. Copyright 1976, Structural Engineers Association of California, and reproduced with permission.

Where reinforcing steel is to be welded, a chemical analysis of the steel shall be provided.[d] Welding shall *conform to "Structural Welding Code— Reinforcing Steel," AWS D1.4-79.*

(D) Flexural Members.

1. General. Flexural members shall not have a width-depth ratio of less than 0.3, nor shall the width be less than ten inches (10") nor more than the supporting column width plus a distance on each side of the column of three-fourths the depth of the flexural member. Flexural members framing into columns shall be subject to a rational joint analysis. *(figure 7-2)*

2. Reinforcement. All flexural members shall have a minimum reinforcement ratio, for top and for bottom reinforcement, of $200/f_y$ throughout their length. The reinforcement ratio "p" shall not exceed 0.025.

The positive moment capacity at the face of columns shall be not less than 50 percent of the negative moment capacity provided. A minimum of one-fourth of the larger amount of the negative reinforcement required at either end shall continue throughout the length of the beam. At least two bars shall be provided both top and bottom. *(figure 7-3)*

3. Splices. Tensile steel shall not be spliced by lapping in a region of tension or reversing stress unless the region is confined by stirrup-ties. Splices shall not be located within the column or within a distance of twice the member depth from the face of the column. At least two stirrup-ties shall be provided at all splices. *(figure 7-4)*

4. Anchorage. Flexural members terminating at a column, in any vertical place, shall have top and bottom reinforcement extending, without horizontal offsets, to the far face of a confined concrete region, terminating in a standard 90 degree hook. Length of required anchorage shall be computed beginning at the near face of the column. Length of anchorage in confined regions shall be not less than 56 percent of the development length, but not less than twenty- four inches (24"). *(figure 7-3)*

> EXCEPTION: Where the column resists less than 25 percent of the story-bent shear, at least 50 percent of such top and bottom reinforcement shall be anchored within such column cores and the remainder shall be anchored in regions outside the column core confined as specified herein for columns.

5. Web Reinforcement. Vertical web reinforcement of not less than No. 3 bars shall be provided in accordance with the requirements of ACI 318, except that:

a. Stirrups shall be spaced to resist the ultimate design shear V_u in which

$$V_u \geq \left| \frac{M_u^A + M_u^B}{L_{AB}} \right| + \left| 1.4 V_{D+L} \right| \qquad (7\text{-}3)$$

where M_u^A and M_u^B are ultimate moment capacities of opposite sense (double curvature) at each hinge location of the member and V_{D+L} is the simple span shear. L_{AB} is the distance between M_u^A and M_u^B. Ultimate moment capacities shall be computed without the ϕ factor reduction and assuming the maximum reinforcing yield strength based on 25 percent over specified yield. Ultimate shear capacities shall be computed with the ϕ factor reduction.

---

[b]Formulas have been renumbered such that SEAOC Formula 2-1 is designated as Formula 7-1 in this manual.

[c]ASTM A706 conforms to these provisions.

[d]Chemical analysis is not required for ASTM A706.

b. Stirrups shall be spaced at no more than d/2 throughout the length of the member.

c. Stirrup-ties, at a maximum spacing of not over d/4, 8 bar diameters, 24 stirrup-tie diameters, or twelve inches (12"), whichever is least, shall be provided in the following locations:

At each end of all flexural members, the first stirrup-tie shall be located not more than two inches (2") from the face of the column and the last, a distance of at least twice the member depth from the face of the columns.

Wherever ultimate moment capacities may be developed in the flexural members under inelastic lateral displacement of the frame.

Wherever required compression reinforcement occurs in the flexural members.

d. In regions where stirrup-ties are required, longitudinal bars shall have lateral support conforming to the provisions of ties for tied columns. Single or overlapping stirrup-ties and supplementary cross-ties may be used.

Section 2(E)

(E) Columns Subject to Direct Stress and Bending.

1. Dimensional Limitations. The ratio of minimum to maximum column thickness shall not be less than 0.4 nor shall any dimension be less than twelve inches (12"). *(figure 7-2)*

2. Vertical Reinforcement. The reinforcement ration "p" in tied columns shall not be less than 0.01 nor greater than 0.06. *(figure 7-3)*

3. Splices. Lap splices shall be made within the center half of column height, and the splice length shall not be less than 30 bar diameters. Continuity may also be effected by welding or by approved mechanical devices provided not more than alternate bars are welded or mechanically spliced at any level and the vertical distances between these welds or splices of adjacent bars is not less than twenty-four inches (24"). *(figure 7-4)*

4. Special Transverse Reinforcement. The cores of columns shall be confined by special transverse reinforcement as specified herein or as required to meet shear requirements. *(figure 7-5)*

a. The volumetric ratio of spiral reinforcement shall not be less than that required in ACI-318 nor

$$p''=0.12\ \frac{f'_c}{f''_{yh}}, \qquad (7\text{-}4)$$

whichever is greater.

b. The total cross-sectional area (A''_{sh}) of rectangular hoop reinforcement shall not be less than

$$A''_{sh}=0.30ah''\ \frac{f'_c}{f''_{yh}}\ \left(\frac{A_g}{A_c}-1\right), \qquad (7\text{-}5)$$

nor

$$A''_{sh}=0.12ah''\ \frac{f'_c}{f''_{yh}}, \qquad (7\text{-}6)$$

whichever is greater, where

a = center to center spacing of hoops in inches with a maximum of four inches (4").

A_c = area of column core.

A_g = gross area of column.
A''_{sh} = total cross-sectional area in square inches of hoop reinforcement including supplementary crossties having a spacing of (a) inches and crossing a section having a core dimension of h".
h" = core dimension of tied column in inches.
f''_{yh} = yield strength of hoop or spiral reinforcement.

Single or overlapping hoops may be provided to meet this requirement. Supplementary crossties of the same size and spacing as hoops using 135 degree minimum hooks engaging the periphery hoop and secured to a longitudinal bar may be used. Supplementary crossties or legs of overlapping hoops shall not be spaced more than fourteen inches (14") on center transversely.

EXCEPTION: Formula (7-5) need not be complied with if the column design is based on the column core only.

c. Special transverse reinforcement shall be provided in that portion of the column over a length equal to the maximum column dimension or one-sixth of the clear height of the column, but not less than eighteen inches (18") from either face of the joint.

d. At any section where the ultimate capacity of the column is less than the sum of the shears (ΣV_u) computed by Formula (7-3) for all the beams framing into the column above the level under consideration, special transverse reinforcement shall be provided. For beams framing into opposite sides of the column, the moment components of Formula (7-3) may be assumed to be of opposite sign. For the purpose of this determination, the factor of 1.4 in Formula (7-3) may be changed to 1.1. For determination of the ultimate capacity of the column, the moments resulting from Formula (7-3) may be assumed to result from deformation of the frame in any one principal axis.

e. Columns which support discontinuous members, such as shear walls, braced frames, or other rigid elements shall have special transverse reinforcement for the full height of the supporting columns.

5. Column Shear. The transverse reinforcement in columns subjected to bending and axial compression shall satisfy the formula

$$A_v f_y \frac{d_c}{s} = \frac{V_u}{\phi} - V_c \qquad (7\text{-}7)$$

where V_u shall be computed by using the ultimate moment capacity in the ends of either the beams or columns framing into the connection. Ultimate moment capacities shall be computed without ϕ or other reduction factors and under all possible vertical loading conditions and assuming the maximum reinforcing yield strength based on 25 percent over specified yield.

$V_c = v_c A_c$, where v_c shall be in accordance with ACI 318, except that V_c shall be considered zero where $\frac{P_u}{A_g} < 0.12 f'_c$.

s = spacing, $\leq$ 1/2 minimum column dimension.
d_c = dimension of column core in direction of load.
A_v = total cross sectional area of special transverse reinforcement in tension within a distance, s, except that two-thirds of such area shall be used in the case of circular spirals.
A_c = Area of column core.

(F) Beam-Column Connection.

1. Analysis. The transverse reinforcement through the connection shall be proportioned according to the requirements of *paragraph 7-3a(1)(E)4*. The transverse reinforcement thus selected shall be checked according to the provisions set forth in *paragraph 7-3a(1)(E)5*, with the exception that V_u, acting on the connection shall be equal to the maximum shears in the connection computed by rational analysis taking into account the column shear and the concentrated shears developed from the forces in the beam reinforcement at a stress assumed at f_y.

Within the depth of the shallowest framing member, special transverse column reinforcement of one-half the amount in the preceding paragraph shall be required where members frame into all four sides of a column and whose width is at least three-fourths the column width. When a corner of a tied column, unconfined by flexural members, exceeds four inches (4"), the full special transverse reinforcement shall be provided through the connection and around bars outside of the connection.

Special transverse beam reinforcing shall be provided through the beam-column connection to provide confinement for longitudinal reinforcement outside the column core where such confinement is not provided by another beam framing into the connection.

2. Design Limitations. At any beam-column connection where $\frac{P_u}{A_g} \geqslant 0.12f'_c$, the total ultimate moment capacity of the column, at the design earthquake axial load, shall be greater than the total ultimate moment capacity of the beams, along the principal planes at that connection.

> EXCEPTION: Where certain beam-column connections at any level do not comply with the above limitations, the remaining columns and connected flexural members shall comply and further shall be capable of resisting the entire shear at that level accounting for the altered relative rigidities and torsion resulting from the omission of elastic action of the non-conforming beam-column connections.

(G) Inspection.

For buildings designed under this Section, a specially qualified inspector shall provide continuous inspection of the placement of the reinforcement and concrete and report to the registered professional engineer responsible for the structural design. The inspector shall submit to the appropriate authority a certificate indicating compliance with the plans and specifications.

(2) Summary of Major SEAOC Modifications to ACI 318-77:

(a) Limitations of precast concrete members (para 7-3a(1)(A)).

(b) Modification to design load factors (para (A), formula 7-1).

(c) Limitations on grades of reinforcing steel (para 7-3a(1)(C)2).

(d) Limitations are placed on dimensions and maximum percentage of reinforcing that can be used (para 7-3a(1)(D)1,2).

(e) Special requirements for splices, anchorages, beam stirrups, column ties and hoops, and joint reinforcement (para 7-3a(1)(D)3, 4, (E), (F)).

(f) Special requirements to provide the formation of inelastic hinges in beams rather than in columns (para 7-3a(1)(E)4d).

(g) The provisions of paragraph 7-3a(1) are illustrated in figures 7-2 through 7-9.

(3) *Special modifications*

(a) Prestressed, post-tensioned, and flat-slab systems are not to be used as part of the lateral force resisting space frame (see para 7-3b for general discussion).

(b) Column ties will be at least No. 4 bars for vertical bars No. 11 or larger and for bundled bars and at least No. 3 bars for vertical bars less than No. 11.

b. General Discussion. Ductility of reinforced concrete frames is accomplished by: (1) using the method of design outlined in ACI 318-77 with a modified load factor, (2) limiting the percentage of steel reinforcement so that the steel will yield before the concrete fails in compression, (3) confinement of the concrete with special transverse reinforcement so as to prevent failure of joints under moment reversals (refer to ACI-352),[a] (4) proportioning members so that any yielding will be confined to the flexural members (girders) rather than to the columns, and (5) avoidance of shear failure. The standard acceptable method of construction for the framing members and their connection is cast-in-place monolithic reinforced concrete. It is sometimes feasible to precast beam-column elements and join them at points of minimum moment with a cast-in-place splice, so an exception is permitted (para 7-3*a*(1)(A)). However, the use of prestressing to develop ductile moment capacity is a subject for further study and is not presently permitted. The use of flat slabs to develop ductile moment capacity is also doubtful, thus does not qualify without special design provisions to provide an equivalent ductile frame within the depth of the slab. Other members within the building, not part of the concrete ductile moment resisting space frame, may be precast, prestressed, composite, or any other appropriate system if adequate diaphragms and connections are developed so the building will respond to seismic input as a unit. These members shall comply with the design requirements of the ACI Building Code, ACI 318.

7-4. Concrete Moment Resisting Space Frames—Concrete Frame Types B and C

a. Concrete Frame Type B. The criteria used to design Type B concrete moment resisting space frames will be ACI 318-77 except appendix A, and as modified below and illustrated in figure 7-10 through figure 7-15. Refer to chapter 3, paragraph 3-6 and table 3-7, for the limitations on the use of this type of concrete space frame.

(1) The provisions of paragraphs 7-3*a*(1)(A), (B), and (D) 1 will apply (see fig 7-2).

(2) Prestressed, post-tensioned, and flat slab systems are not to be used as part of the lateral force resisting space frame (see para 7-3*b*).

(3) The specified yield strength of reinforcing steel will not exceed 60,000 p.s.i.

(4) Members of the moment resisting space frame will be designed for the shear that results from the formation of inelastic joint rotations, in the same direction, at each end of the member (see fig 7-14).

[a]Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," ACI Journal, *Proceedings* V. 73, No. 28, July 1976. This reference provides a state-of-the-art summary of current information.

(5) All frame flexural members will have a m mum reinforcement ratio, p, for top and bott reinforcement of $200/f_y$ throughout their length except where a greater minimum is required by ACI 318. At least two bars will be provided, both top and bottom, throughout their length.

(6) At locations where the ultimate capacity of a member will be developed under inelastic lateral displacement of the frame, the maximum p will not exceed 0.025.

(7) The positive moment capacity of flexural members at columns will be at least 40 percent of the negative moment capacity.

(8) Splices in required reinforcing of flexural members framing into columns will not be located within the column nor within a distance of twice the member depth from the face of the column. At least two closed stirrup ties will be provided at all splices.

(9) Flexural member framing into a column where there is no flexural member on the opposite side will have top and bottom reinforcement extending to the far face of the confined region and terminated with a standard hook.

(10) The length of anchorage in confined regions may be 0.56 l_d. In other regions, anchorage length will be l_d. In no case will the anchorage length be less than 24 inches (l_d is developm length per ACI).

(11) Stirrup ties of not less than No. 3 bars will be provided at a spacing of not over d/4 nor 12" for a distance of at least the member depth at the end of each flexural member and wherever ultimate capacities may be reached under lateral displacement of the frame. The first stirrup tie will be placed 2" from the face of the column.

(12) Standard stirrups will be provided at a maximum spacing of 3/4d throughout the length of the flexible member, or minimum required by ACI 318, whichever governs.

(13) The reinforcement ratio, p, in tied columns will not be less than 0.01 nor greater than 0.06.

(14) Lap splices shall be made within the center half of column height, and the splice length shall not be less than 30 bar diameters. Continuity may also be effected by welding or by approved mechanical devices provided not more than alternate bars are welded or mechanically spliced at any level and the vertical distance between these welds or splices of adjacent bars is not less than twenty-four inches (24").

(15) Special transverse reinforcement for columns will be continuous reinforcement enclosing longitudinal reinforcement and ending with a degree bend with a 10 bar diameter extension. Sup

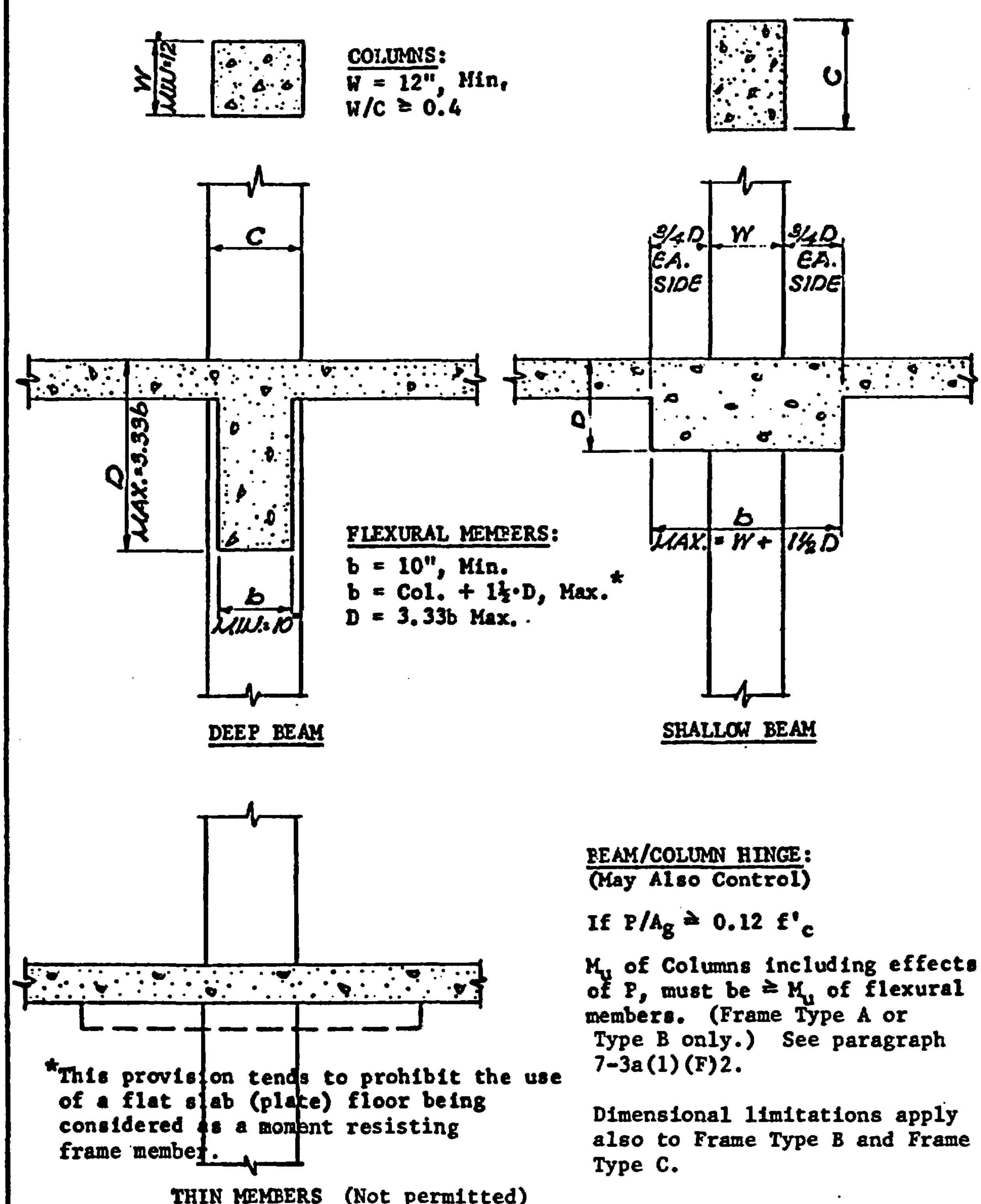
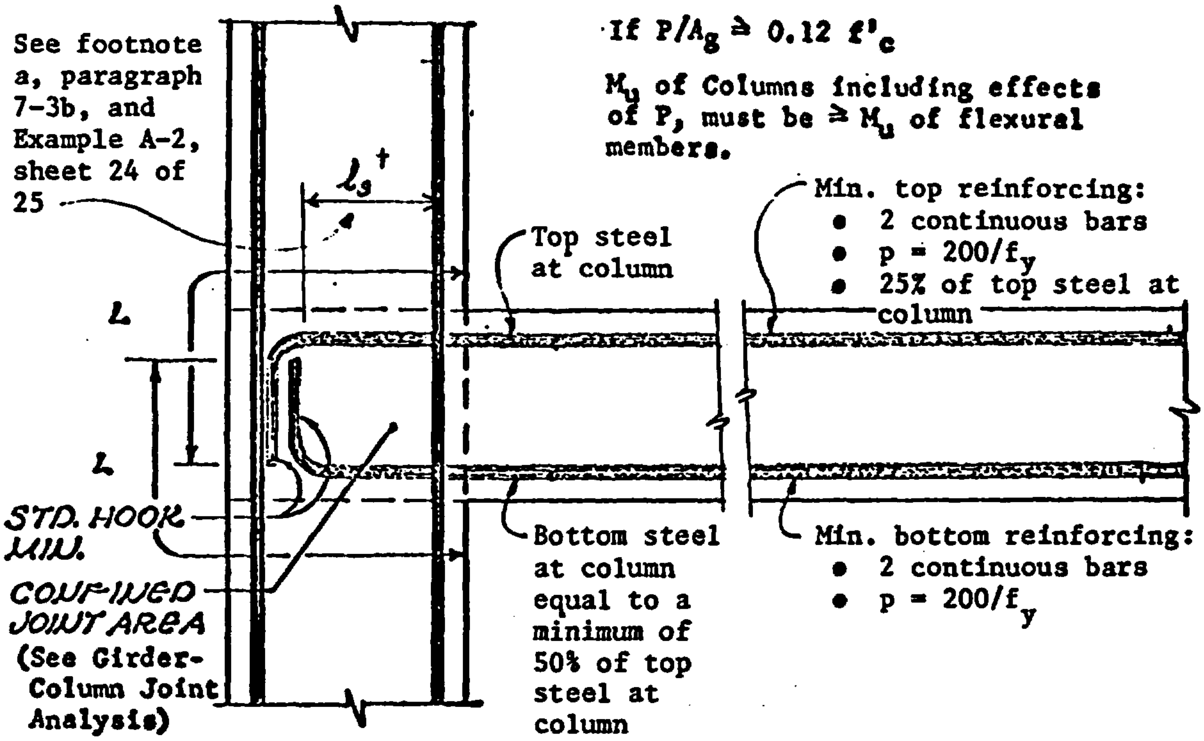


Figure 7-2. Concrete Frame Type A - Limitations on Dimensions



ELEVATION OF GIRDER AND COLUMN

FLEXURAL MEMBER:

$f'_c = 3{,}000$ p.s.i. min. at 28 days

$f_y = 40$ ksi (ASTM A615) or 60 ksi (ASTM A615 or ASTM A706)
Reinforcement ratio, $p = A_s/bd$ or $p' = A'_s/bd$: $p = 0.025$ max.

†l_s is not a code requirement. It is a recommendation of ACI Committee 352, ACI Journal, July 1976.

L (Anchorage) $= 0.56\ l_d$* OR Min. 24"

*L = l_d for Top Member (without column above)
NOTE: For l_d, development length of deformed bars in tension, see ACI 318-77, Sect. 12.2.

COLUMN:

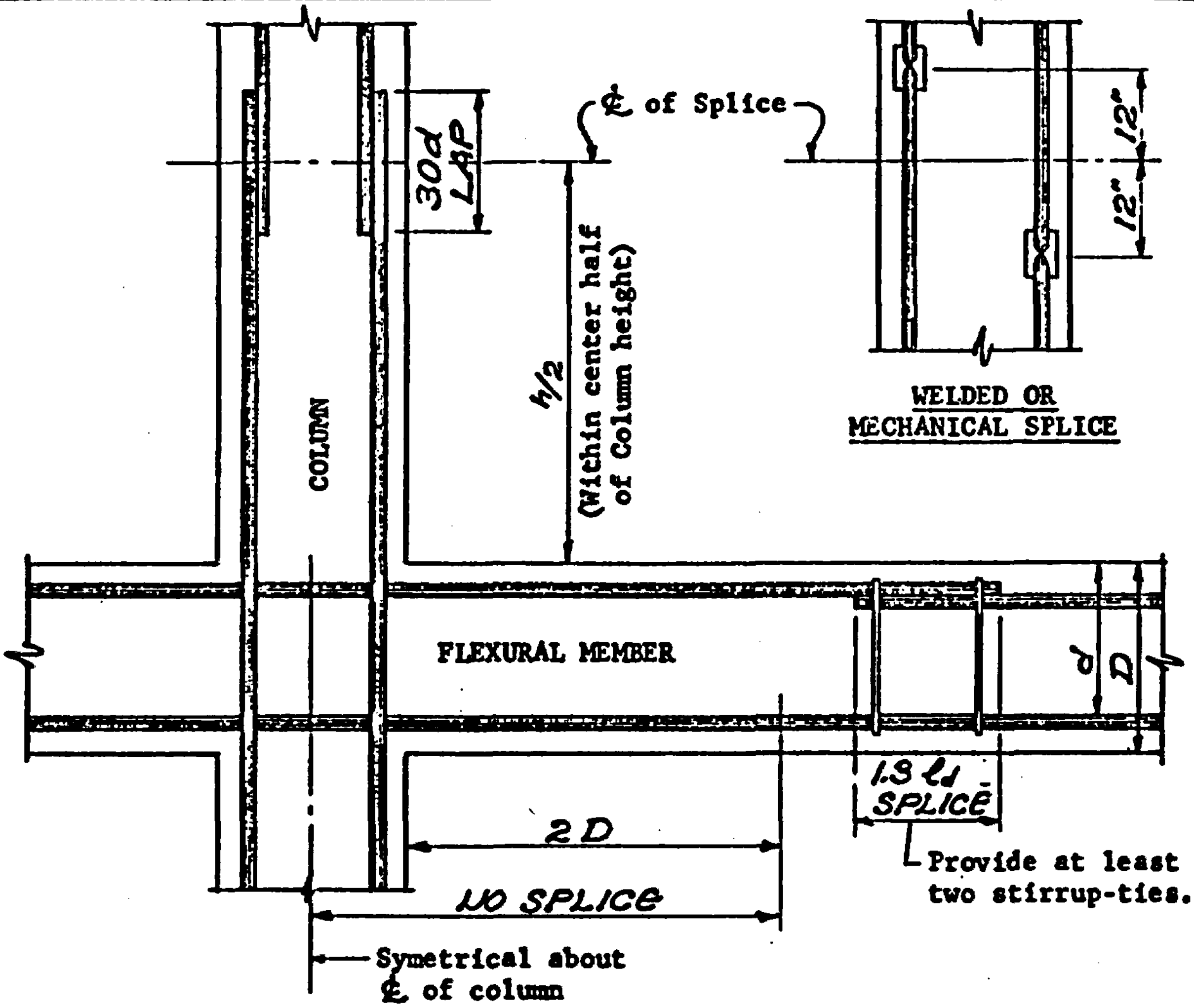
$f'_c = 3{,}000$ p.s.i. at 28 days Min.

$f_y = 40$ ksi (ASTM A615) or 60 ksi (ASTM A615 or ASTM A706)

Reinforcement ratio, p (for tied columns)
≥ 0.01 and ≤ 0.06.

Reference:
paragraph 7-3

Figure 7-3. Concrete Frame Type A - Longitudinal Reinforcement



COLUMN:

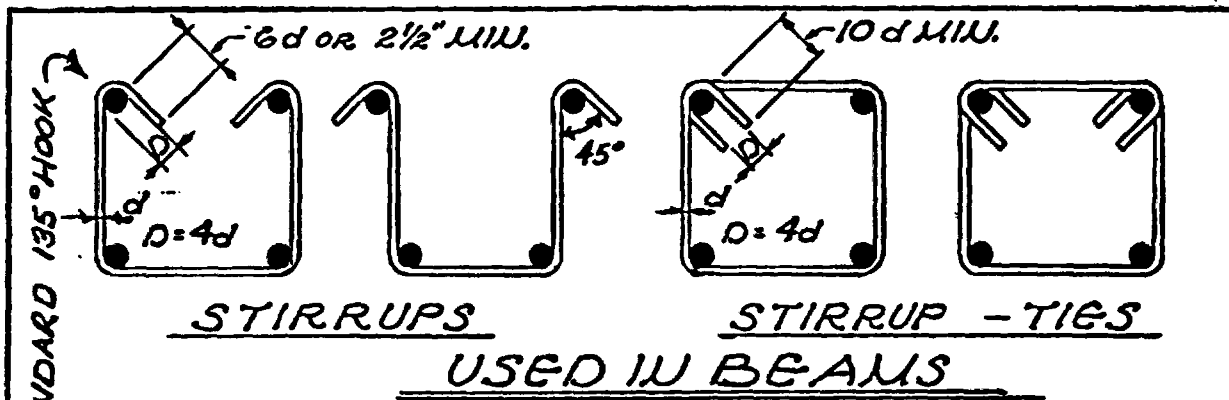
l_d is the tension development length. See ACI 318-77, Sect. 12.2.

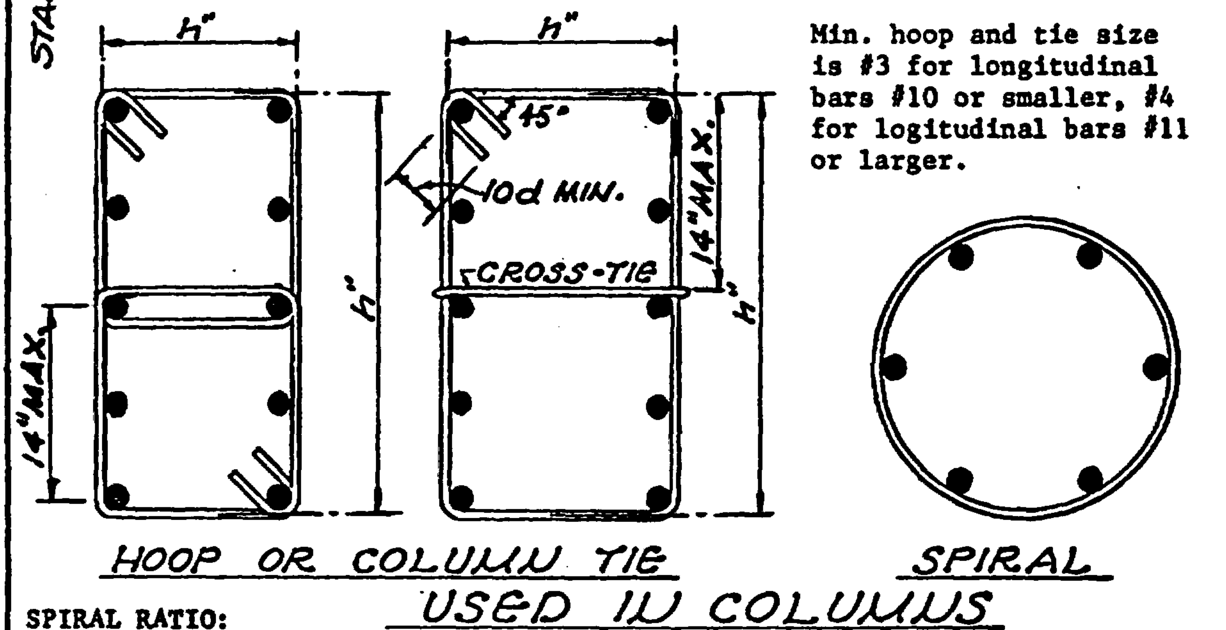
At any level, not more than alternate bars will be welded or mechanical spliced. Minimum distance between two adjacent bar splices = 24".

For #14S & #18S bars, welded splices are recommended. Lap splices will not be used.

Reference:
paragraph 7-3

Figure 7-4. Concrete Frame Type A - Splices in Reinforcement





Min. hoop and tie size is #3 for longitudinal bars #10 or smaller, #4 for logitudinal bars #11 or larger.

SPIRAL RATIO:

$$\rho_s = 0.45 \frac{f'_c}{f_y}\left(\frac{A_g}{A_c} - 1\right) \text{ or } 0.12 \frac{f'_c}{f''_{yh}},$$

whichever is greater.

HOOP REQUIREMENTS - TOTAL TIE AREA:

$$A''_{sh} = 0.30\ ah'' \frac{f'_c}{f''_{yh}}\left(\frac{A_g}{A_c} - 1\right) \text{ or}$$

Formula 7-5
Formula 7-6

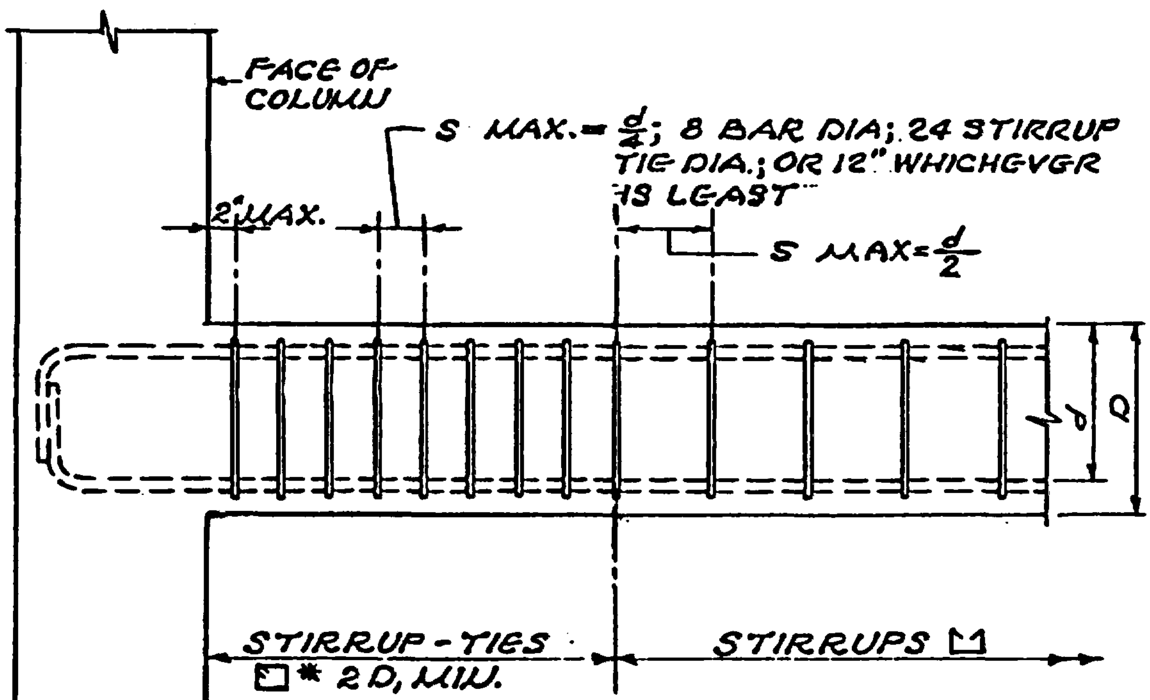
$$A''_{sh} = 0.12 \frac{f'_c}{f''_{yh}}, \text{ whichever is greater.}$$

FUNCTIONS	Stirrups	Stirrup-Ties	Column Ties	Hoops	Spirals
Shear Reinforcement And "Caging"	●	●	●	●	●
Restrain Longitudinal Steel From Buckling		●	●	●	●
Confine Concrete				●	●

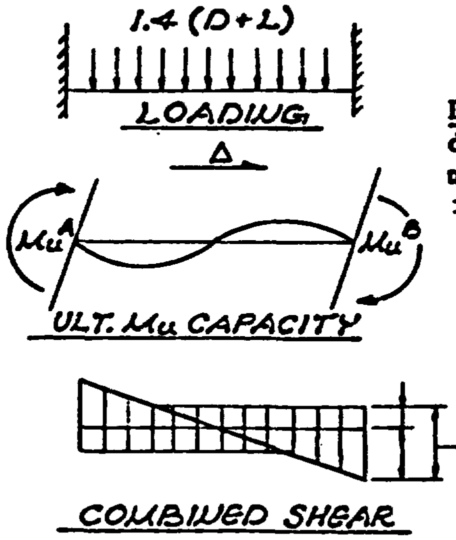
Provide hoops or spirals in columns where special transverse reinforcement is required. See paragraph 7-3a(1)(E)4.

Reference: paragraph 7-3

Figure 7-5. Concrete Frame Type A - Transverse Reinforcement



* STIRRUP - TIES REQUIRED (#3 MIN.) EACH END; WHERE M_u MAY BE DEVELOPED; WHERE REQUIRED COMPRESSION STEEL OCCURS; AND AT ALL SPLICES.



$$V_u^B \geq \frac{M_u^A + M_u^B}{L} + 1.4\,V_{D+L}$$

DESIGN: Vertical web reinforcement in accordance with Chapter 11 of ACI 318, and the minimum requirements in paragraph 7-3a(1)(D) as indicated herein.

NOTE: M_u^A & M_u^B = Ultimate moment capacity of opposite sense, at each end of the member, computed without the ϕ factor reduction and assuming the maximum reinforcing yield strength based on 25% over specified yield.

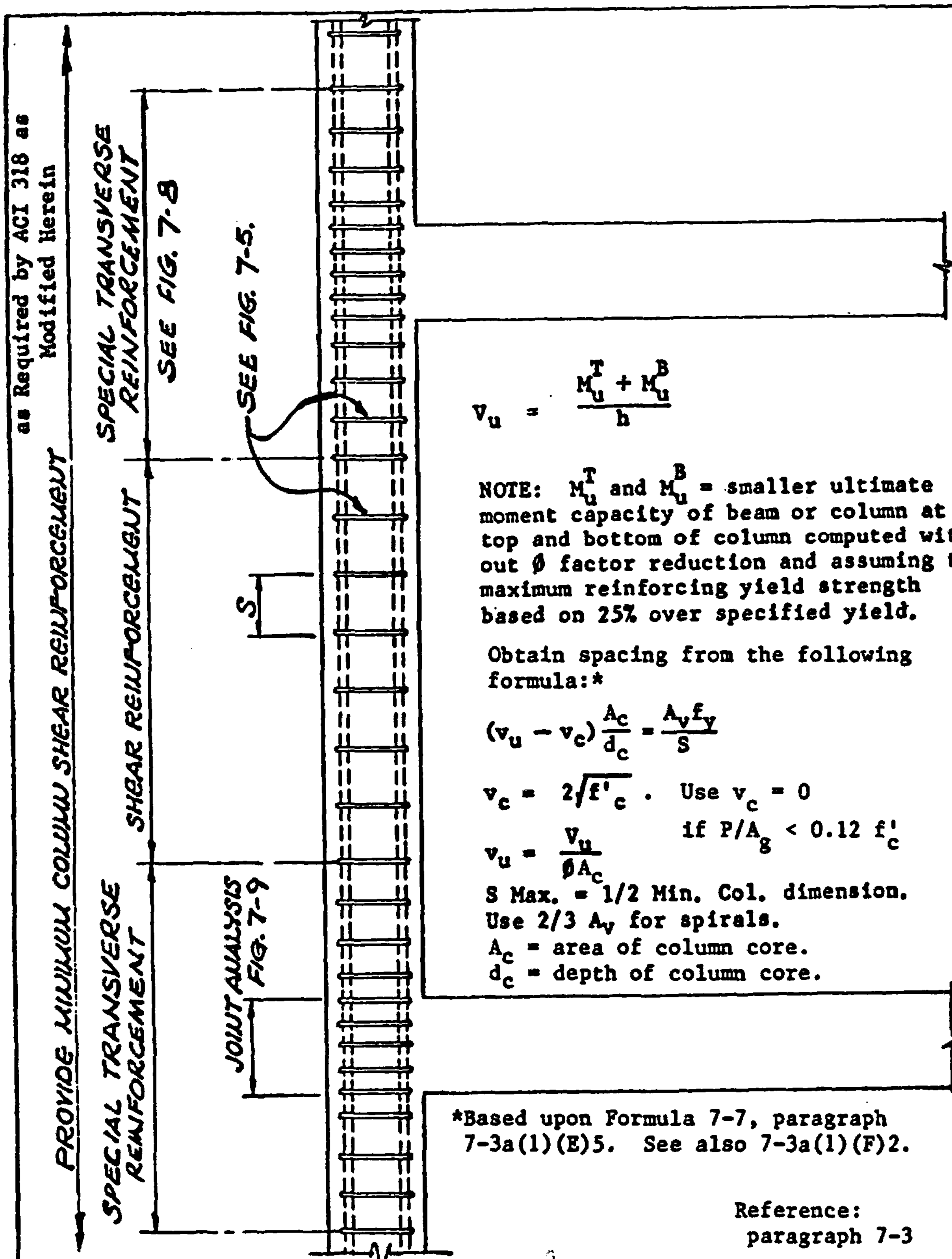
$$(v_u - v_c) = \frac{A_v f_y}{s b}$$

$$v_c = 2\sqrt{f'_c} \qquad v_u = \frac{V_u}{\phi b d}$$

$$v_u \ MAX. = 10\sqrt{f'_c}$$

Reference:
paragraph 7-3

Figure 7-6. Concrete Frame Type A - Girder Web Reinforcement



$$v_u = \frac{M_u^T + M_u^B}{h}$$

NOTE: M_u^T and M_u^B = smaller ultimate moment capacity of beam or column at top and bottom of column computed without $\emptyset$ factor reduction and assuming the maximum reinforcing yield strength based on 25% over specified yield.

Obtain spacing from the following formula:*

$$(v_u - v_c)\frac{A_c}{d_c} = \frac{A_v f_y}{S}$$

$v_c = 2\sqrt{f'_c}$. Use $v_c = 0$ if $P/A_g < 0.12\ f'_c$

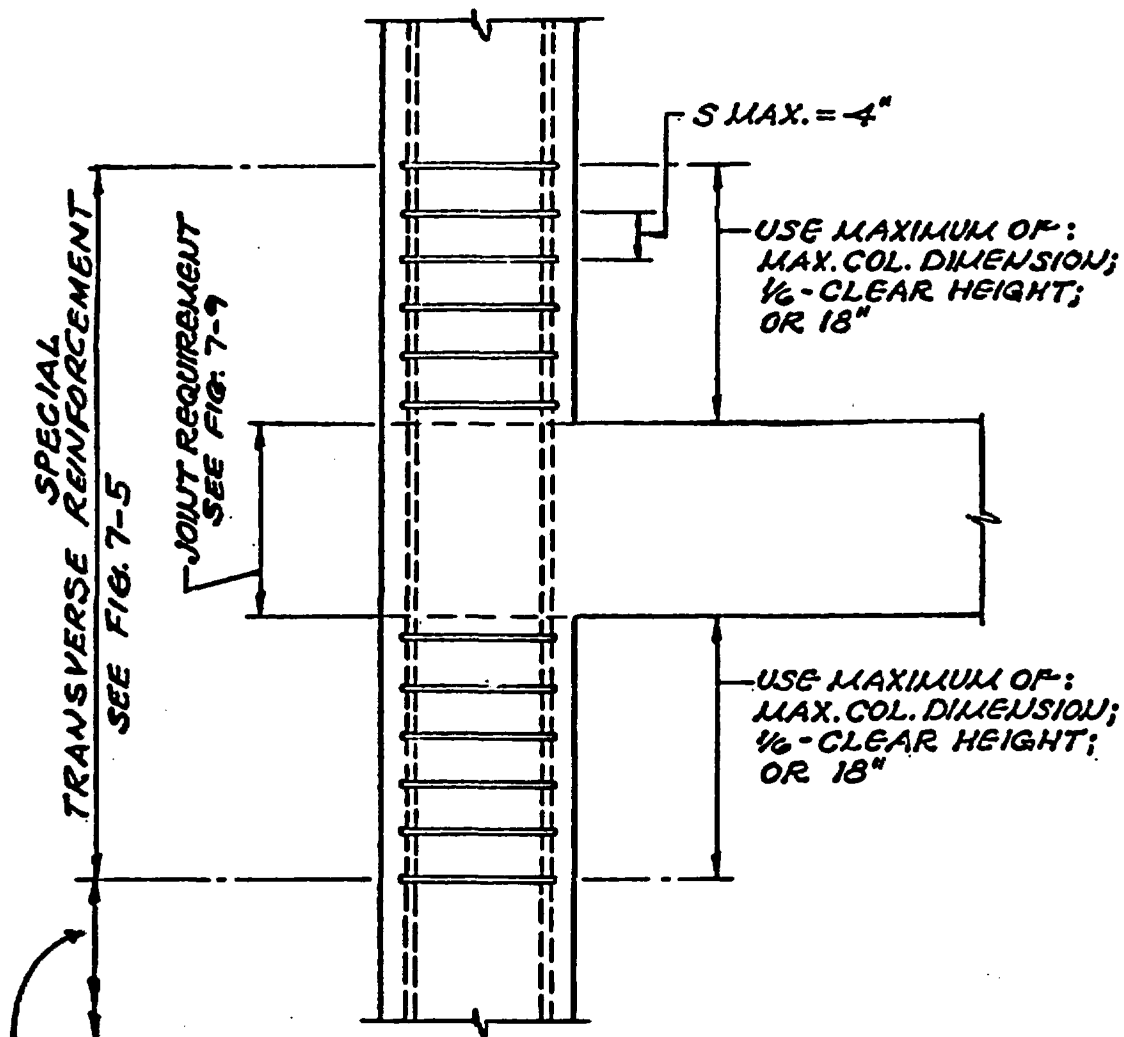
$$v_u = \frac{V_u}{\emptyset A_c}$$

S Max. = 1/2 Min. Col. dimension.
Use 2/3 A_v for spirals.
A_c = area of column core.
d_c = depth of column core.

*Based upon Formula 7-7, paragraph 7-3a(1)(E)5. See also 7-3a(1)(F)2.

Reference:
paragraph 7-3

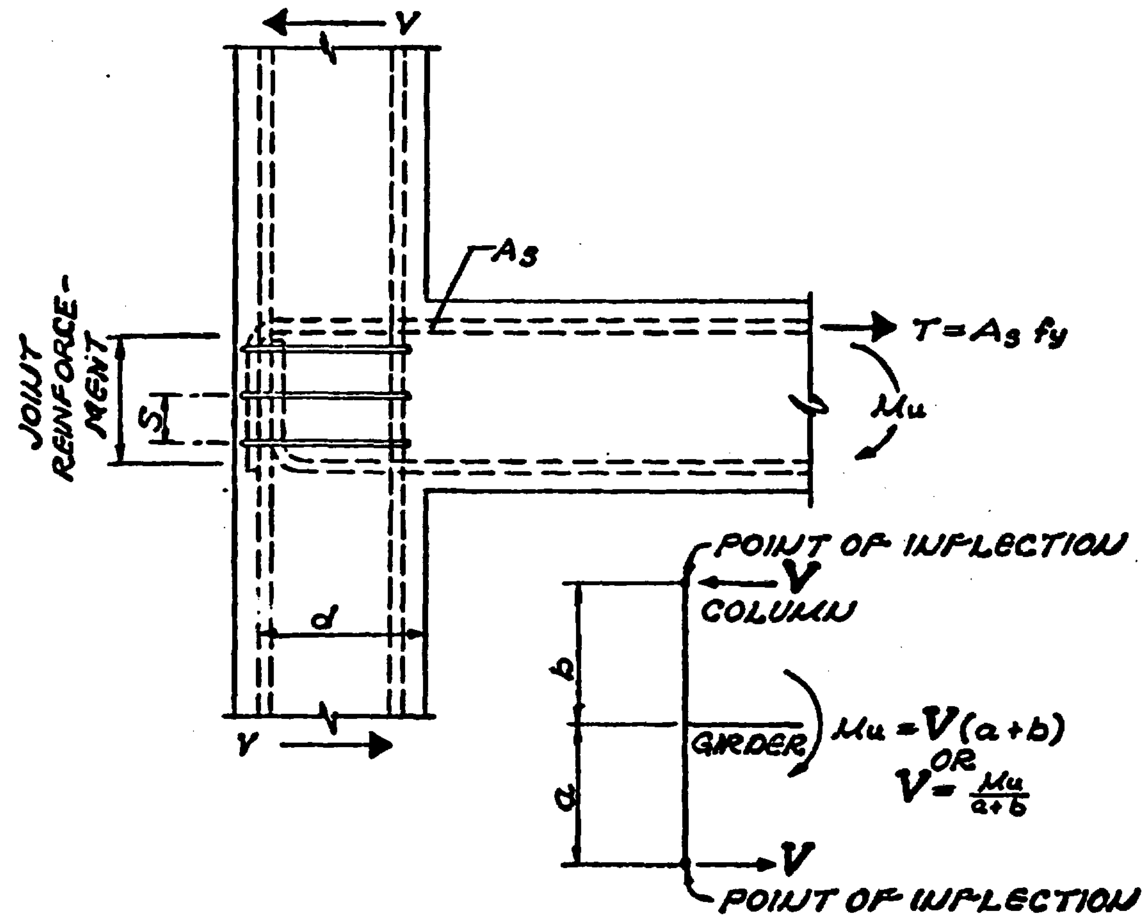
Figure 7-7. Concrete Frame Type A - Column Transverse Reinforcement



AT ANY SECTION WHERE THE ULTIMATE CAPACITY OF THE COLUMN (P_u) IS LESS THAN THE SUM OF THE SHEARS (ΣV_u) COMPUTED BY $V_u = \frac{M_u^A + M_u^B}{L} + 1.1 V_D + L$ FOR ALL THE BEAMS. ABOVE THE LEVEL UNDER CONSIDERATION, CONFINING REINFORCEMENT SHALL BE PROVIDED. SEE FIG. 7-5. SEE ALSO THE PROVISIONS OF SECT. 7-3a(1)(E)4e.

Reference:
paragraph 7-3

Figure 7-8. Concrete Frame Type A - Special Transverse Reinforcement



$V_u = T - V = A_s f_y - \frac{M_u}{a+b}$ $\quad v_u = \frac{V_u}{\phi b d}$

$S = \frac{A_v f_y}{(v_u - v_c) b}$ WHERE $v_c = 2\sqrt{f'_c}$ EXCEPT WHEN $P/A_g < 0.12 f'_c$ THEN $v_c = 0$

$S = 4''$ MAX.

Only 1/2 the special transverse reinforcement is required for columns where girders frame into all four sides.

NOTE: Column Confining Reinforcement is a minimum and may govern. See Figure 7-5.

The amount of reinforcement at the intersections frequently results in congestion of bars. A careful study of the bar layouts should be made during design.

Reference: paragraph 7-3

Figure 7-9. Concrete Frame Type A - Girder-Column Joint Analysis plementary cross ties, if needed, will have standard hooks at the ends. Single leg cross ties may be lap spliced if a minimum of 20 diameter lap is provided. Refer to figure 7-13.

(16) At the ends of columns, special transverse reinforcement will be provided over a length equal to the maximum column dimension or one-sixth the clear height of the column, but not less than 18" from either face of the joint. This transverse reinforcement will be spaced at not over 4" on center and have a total cross-sectional area of not less than

$A''_{sh}=0.08\,ah''\,\frac{f'_c}{f''_{yh}}$ (see para 7-3a(1)(E)4 for definition of terms)

(17) A minimum special transverse reinforcement of No. 4 at a maximum spacing of 4" on center, or equivalent, will be provided throughout the beam-column joint. The requirement for cross ties (fig 7-13) may be omitted within the joint if the longitudinal column bars are confined by adjoining beams.

b. Concrete Frame Type C. The criteria used to design Type C concrete moment resisting space frames will be ACI 318-77 except appendix A, and as modified below. This type of space frame is limited in use to Seismic Zone 1, for K not less than 1.0, and for buildings not taller than 80 feet, when designed to resist earthquake forces (see chap 3, para 3-6 and table 3-7).

(1) For earthquake loading ACI 318 load factors will be modified to formulas 7-1 and 7-2 in paragraph 7-3*a*(1)(A), and the dimensional limits of paragraph 7-3*a*(1)(D)1 will apply (see fig 7-2).

(2) Flexural members are required to have web reinforcement throughout the length of the member. It will be designed in accordance with ACI-318 except that such web reinforcement shall not be less than 0.0015 times the product of the width of the web and the spacing of the web reinforcement along the longitudinal axis of the member. The first stirrup will be located at 2 inches from the column face. The next six stirrups will be placed not over d/4.

(3) Positive moment reinforcement at the supports of flexural members subject to reversal of moments will be anchored by bond, hooks, or mechanical anchors in or through the supporting member to develop the yield strength of the bar. The positive moment capacity of flexural members at columns will be at least 30 percent of the negative capacity.

(4) Lapped splices in flexural members, located in a region of tension or reversing stress, will be confined by at least two stirrups at each splice.

(5) The spacing of ties at the ends of tied columns will not exceed 4 inches for a distance equal to the maximum column dimension but not less than one-sixth of the clear height of the column from the face of the joint. The first such tie will be located 2 inches from the face of the joint. Joints of exterior and corner columns will be confined by lateral reinforcement through the joint. Such lateral reinforcement will consist of spirals or ties as required at the ends of columns.

7-5. Steel ductile moment resisting space frames—Steel Frame Type A.

a. General Design Criteria. The criteria used to design steel ductile moment resisting space frames will be the latest edition of AISC Specification as modified by SEAOC Section 4 (reprinted below).

(1) SEAOC Section 4, Steel Ductile Moment-Resisting Space Frames:*

(A) General.

Design and construction of steel framing in ductile moment resisting space frames shall conform to the latest edition of the American Institute of Steel Construction "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings" and the American Welding Society's "Structural Welding Code" AWS D1.1 latest edition and to all the requirements of this Section.

(B) Definitions.

CONNECTION consists of only those elements that connect the member to the joint.

JOINT is the entire assemblage at the intersections of the members.

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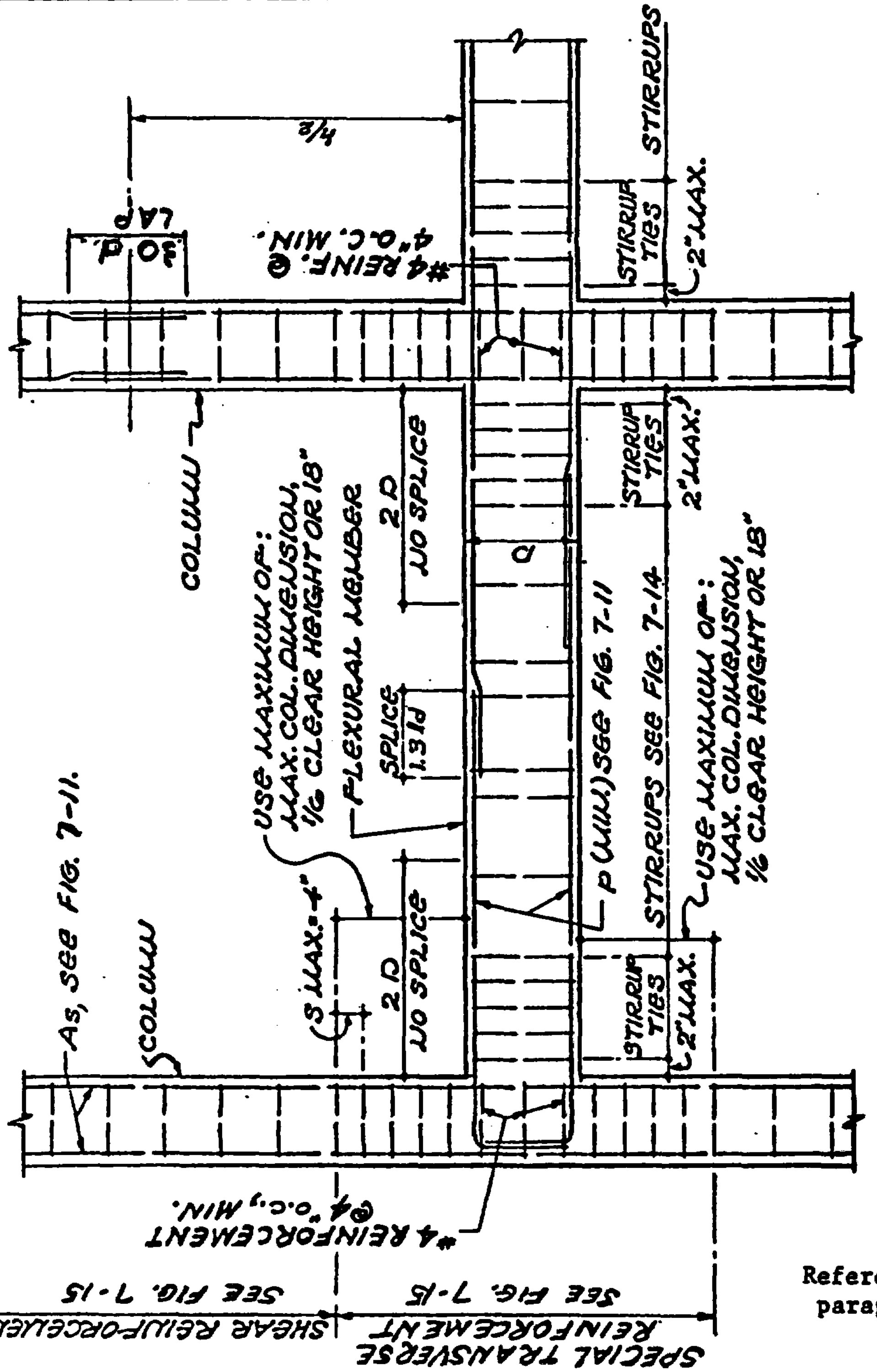
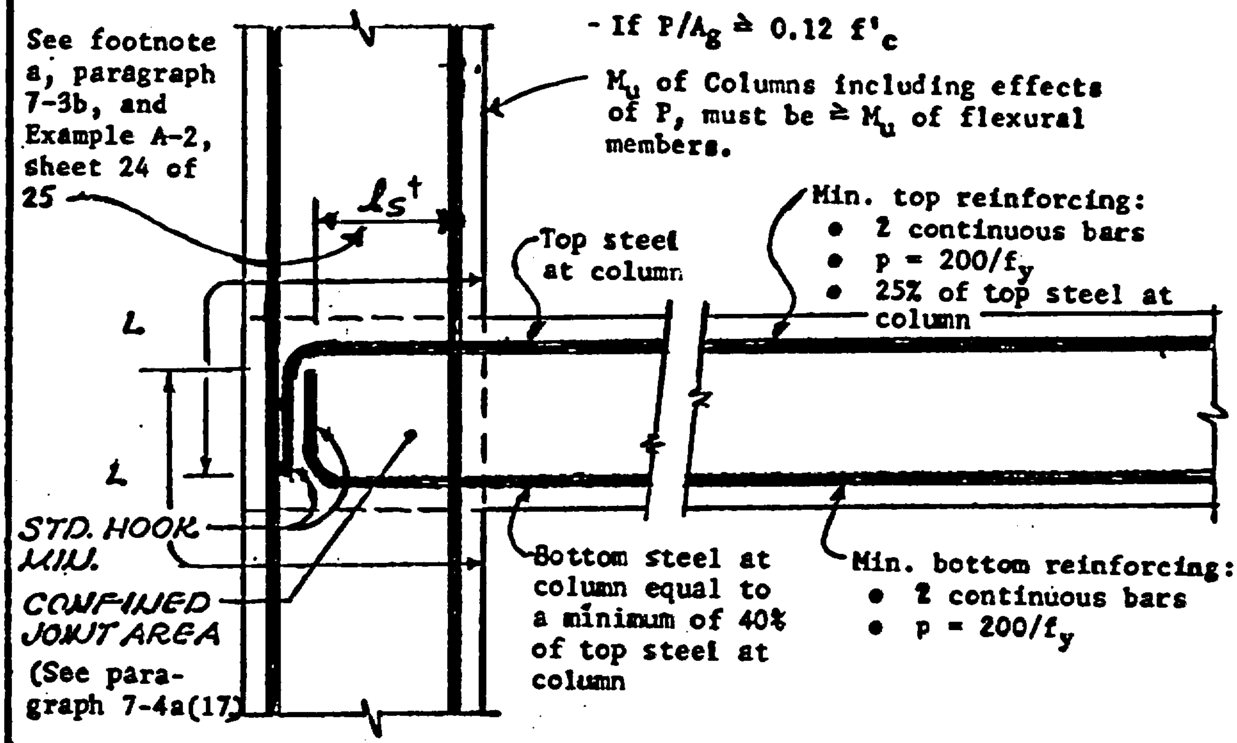


Reference:
paragraph 7-4

Figure 7-10. Concrete Frame Type B - Frame Requirements



ELEVATION OF GIRDER AND COLUMN

FLEXURAL MEMBER:

f'_c = 3,000 p.s.i. min. at 28 days

f_y = 40 ksi (ASTM A615) or 60 ksi (ASTM A615 or A706).

Reinforcement ratio $p = A_s/bd$ or $p' = A'_s/bd$: $p = 0.025$ max.

†l_s is not a code requirement. It is a recommendation of ACI Committee 352, ACI Journal, July 1976.

L (Anchorage) = 0.56 l_d* OR Min. 24"

L = l_d for Top Member (without column above)

*NOTE: For l_d, development length of deformed bars in tension, see ACI 318-77, Sect. 12.2.

COLUMN:

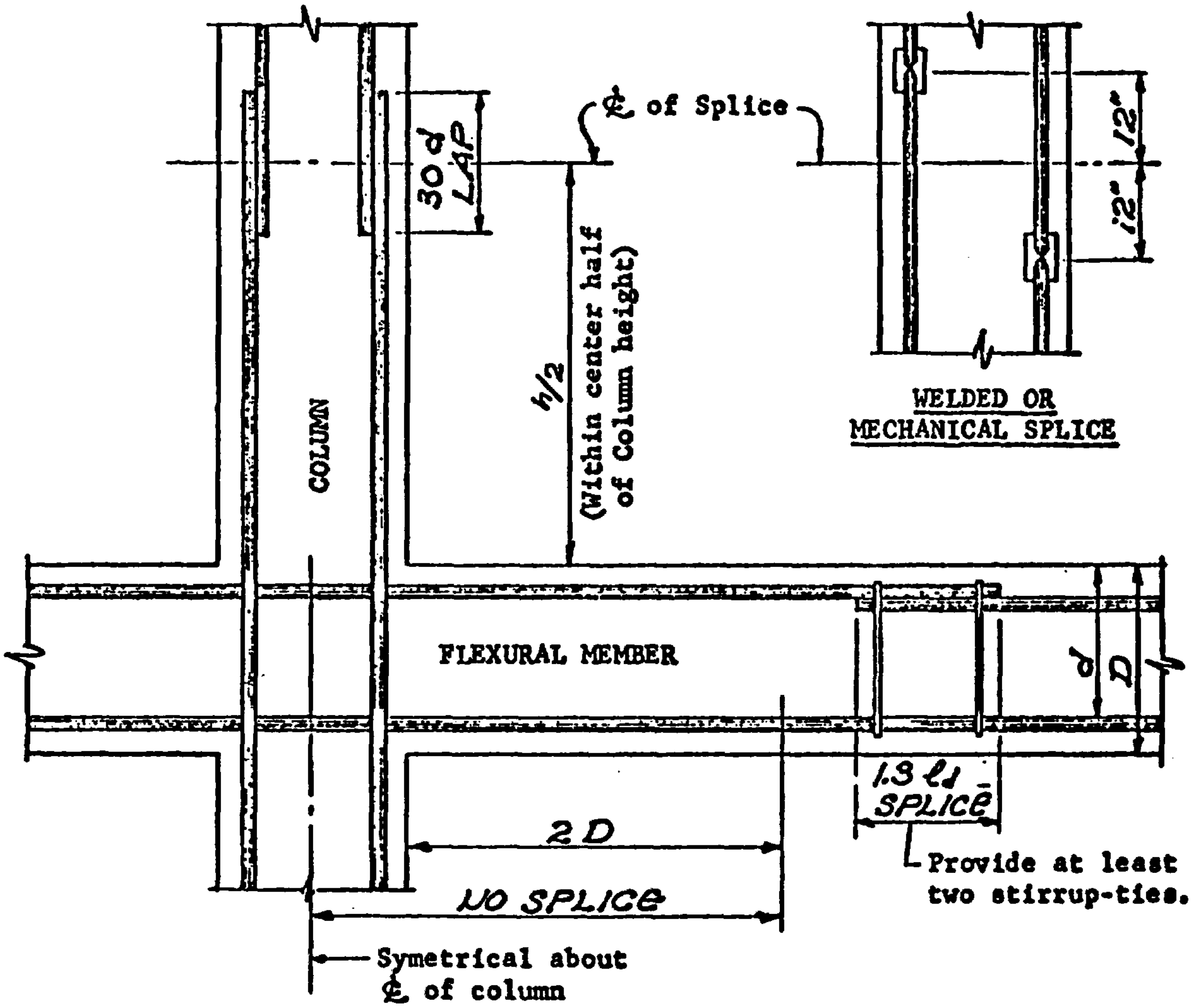
f'_c = 3,000 p.s.i. at 28 days Min.

f_y = 40 ksi (ASTM A615) or 60 ksi (ASTM A615 or ASTM A706)

Reinforcement ratio, p (for tied columns) ≥ 0.01 and ≤ 0.06.

Reference: paragraph 7-4

Figure 7-11. Concrete Frame Type B - Longitudinal Reinforcement



COLUMN:

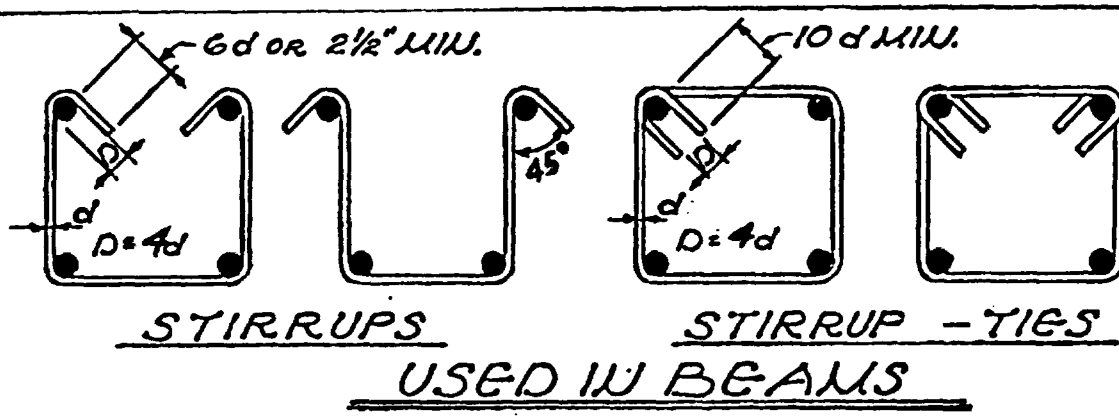
l_d is the tension development length. See ACI 318-77, Sect. 12.2.

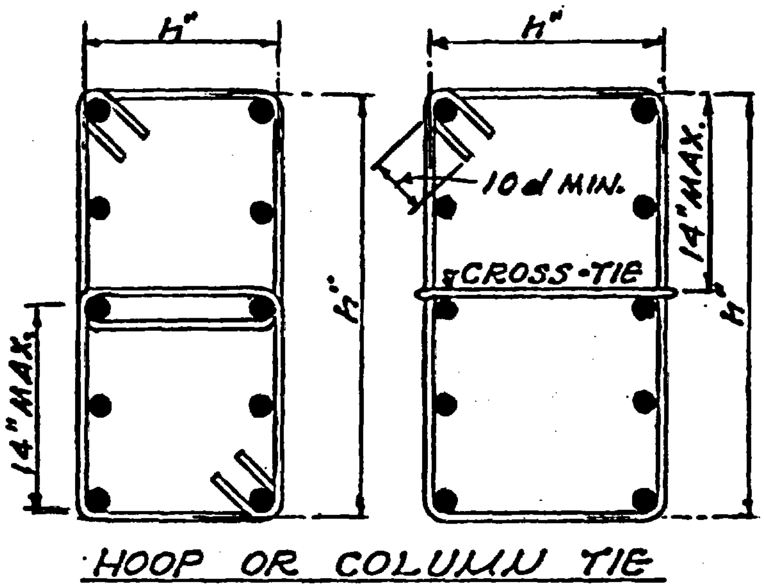
At any level, not more than alternate bars will be welded or mechanical spliced. Min. distance between two adjacent bar splices = 24".

For #14S & #18S bars, welded splices are recommended. Lap splices will not be used.

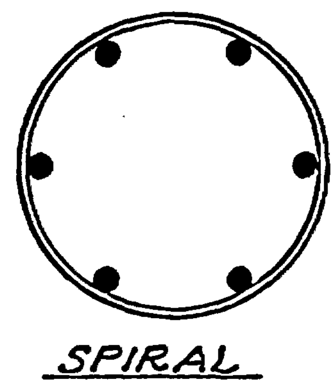
Reference:
paragraph 7-4

Figure 7-12. Concrete Frame Type B - Splices in Reinforcement





Min. hoop and tie size is #3 for longitudinal bars #10 and smaller, and #4 for longitudinal bars #11 or larger.



USED IN COLUMNS

SPIRAL RATIO:

$$\rho_s = 0.45 \frac{f'_c}{f_y}\left(\frac{A_g}{A_c} - 1\right) \text{ or } .08 \frac{f'_c}{f_y}$$

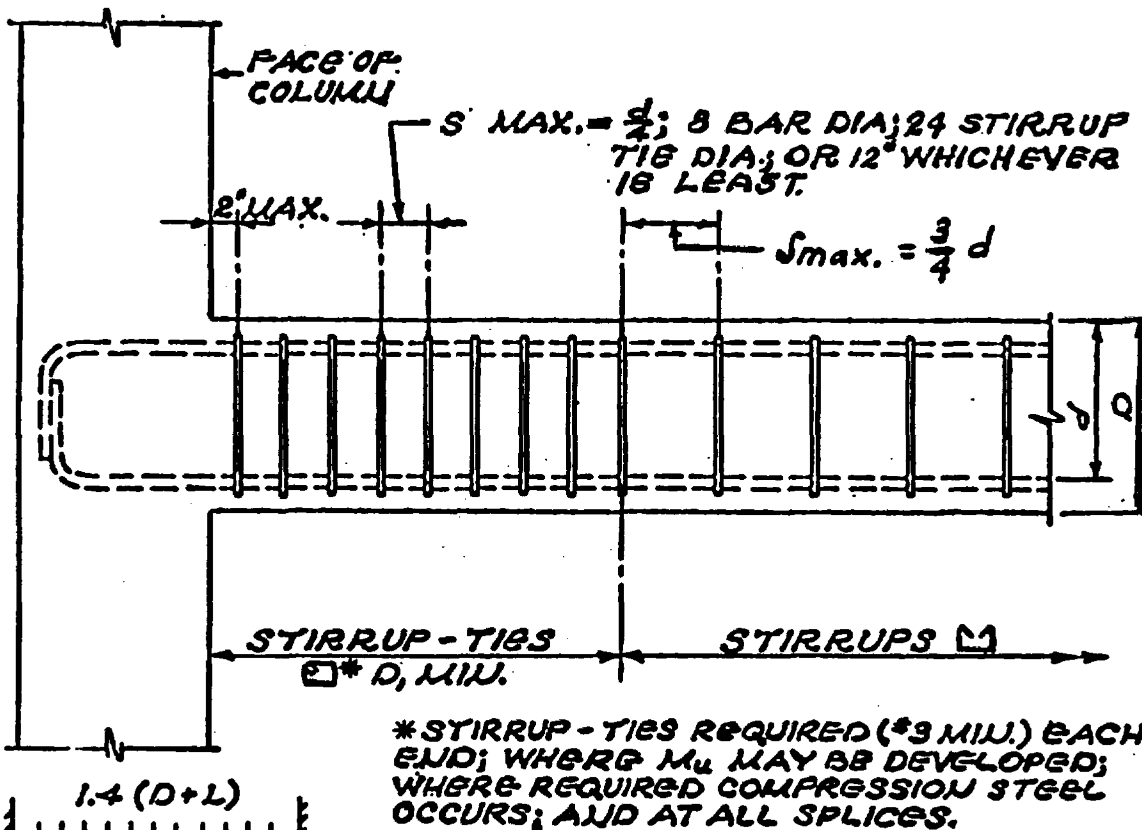
HOOP REQUIREMENTS - TOTAL TIE AREA:

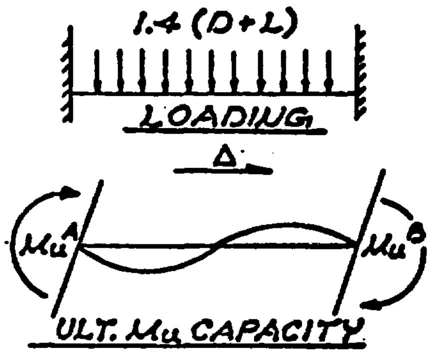
$$A''_{sh} = 0.08\, ah'' \frac{f'_c}{f''_{yh}}$$

FUNCTIONS	Stirrups	Stirrup-Ties	Column Ties	Hoops	Spirals
Shear Reinforcement And "Caging"	●	●	●	●	●
Restrain Longitudinal Steel From Buckling		●	●	●	●
Confine Concrete				●	●

Reference:
paragraph 7-4

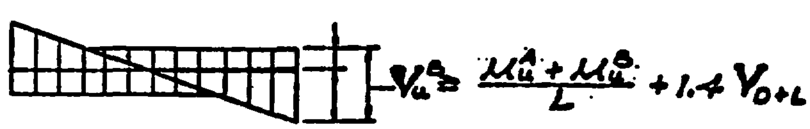
Figure 7-13. Concrete Frame Type B - Transverse Reinforcement





DESIGN: Vertical web reinforcement in accordance with Chapter 11 of ACI 318, and the minimum requirements in paragraph 7-3a(1)(D).

NOTE: M_u^A & M_u^B = Ultimate moment capacity of opposite sense, at each end of the member.



COMBINED SHEAR

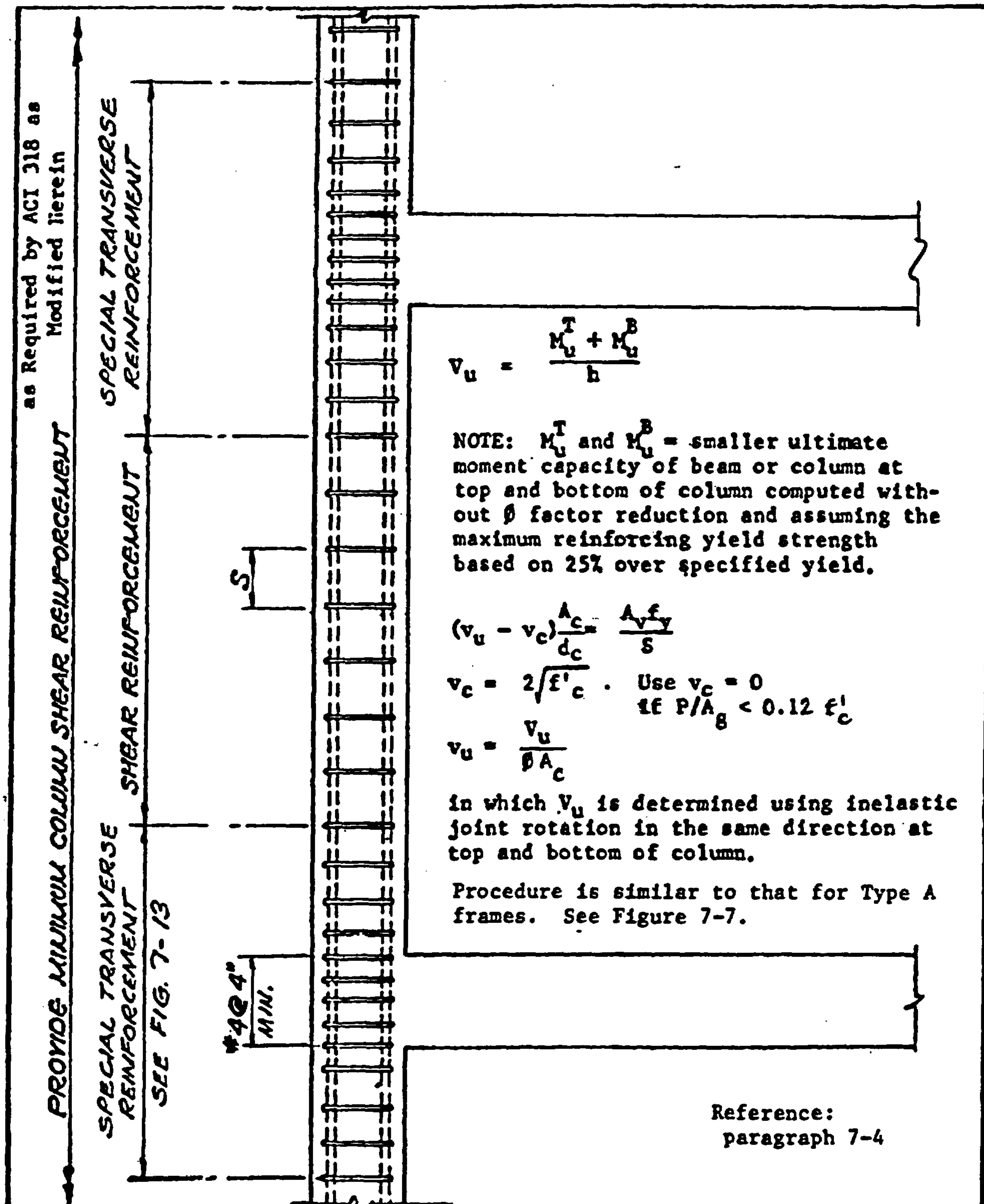
$$v_u - v_c = \frac{A_v f_y}{Sb}$$

$$v_c = 2\sqrt{f_c'} \qquad v_u = \frac{V_u}{\phi b d}$$

$$v_u \text{ MAX.} = 10 \sqrt{f_c'}$$

Reference:
paragraph 7-4

Figure 7-14. Concrete Frame Type B - Girder Web Reinforcement



$$V_u = \frac{M_u^T + M_u^B}{h}$$

NOTE: M_u^T and M_u^B = smaller ultimate moment capacity of beam or column at top and bottom of column computed without ϕ factor reduction and assuming the maximum reinforcing yield strength based on 25% over specified yield.

$$(v_u - v_c)\frac{A_c}{d_c} = \frac{A_v f_y}{S}$$

$$v_c = 2\sqrt{f'_c} \ . \quad \text{Use } v_c = 0 \text{ if } P/A_g < 0.12\ f'_c$$

$$v_u = \frac{V_u}{\phi A_c}$$

in which V_u is determined using inelastic joint rotation in the same direction at top and bottom of column.

Procedure is similar to that for Type A frames. See Figure 7-7.

Reference:
paragraph 7-4

Figure 7-15. Concrete Frame Type B — Column Transverse Reinforcement

(C) **Materials.**

Structural steel shall conform to one of the following ASTM Specifications, latest edition: *A36, A441, A500 (Grades B and C), A501, A572 (Grades 42, 45, 50 and 55), or A588.* Exceptions: Structural Steel ASTM A283 Grade D may be used for base plates and anchor bolts.

(D) **Connections.**

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.

EXCEPTION: The connection need not develop the full plastic capacity of the beam or girder if it can be shown that adequately ductile joint displacement capacity is provided with a lesser connection.

For steel whose specified ultimate strength is less than 1.5 of the specified yield strength, plastic hinges in beams formed during inelastic deformations of the frame shall not occur at locations in which the beam flange area has been reduced such as by holes for bolts.

(E) **Local Buckling.**

Members in which hinges will form during inelastic displacement of the frames shall comply with the requirements for "plastic design sections."

(F) **Non-Destructive Weld Testing.**

Tension groove welded connections between primary members of the ductile moment resisting space frame shall be tested by non-destructive methods for compliance with AWS D1.1 and job specifications. A program for this testing shall be established by the engineer.

b. General Discussion.

(1) The beams (or girders) will be connected to columns by rigid joints which are capable of developing in the beams the full plastic capacity of the members framing into the joint, under moment reversals. Members in which hinges will form during inelastic displacement of the frames shall comply with the requirements of the AISC plastic design method.

(2) Additional discussion is included in the SEAOC Commentary on Section 4.

(3) For typical details refer to figure 7-16.

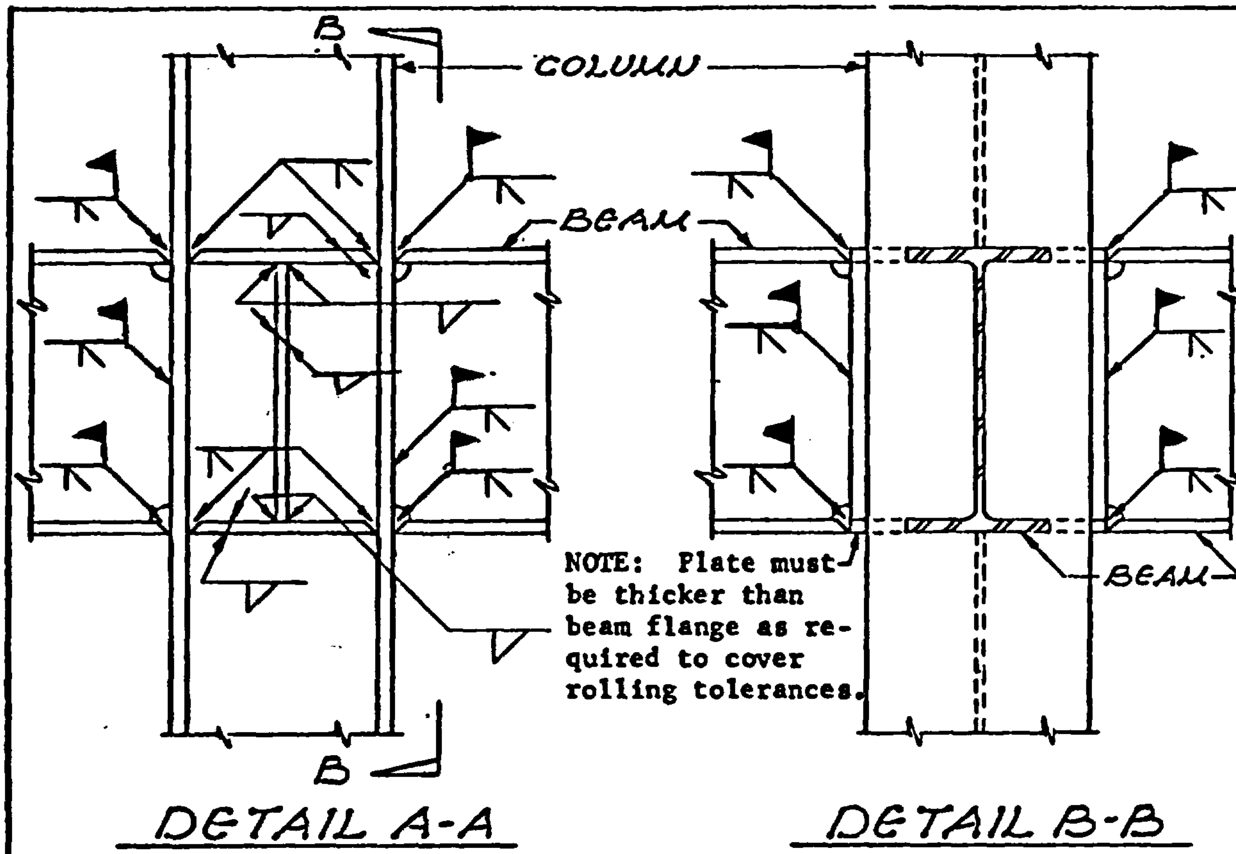
7-6. Steel moment resisting space frames—Steel Frame Types B and C. *a. General Design Criteria.* The criteria used in the design of steel moment resisting space frames will be the latest edition of the specifications of the American Institute of Steel Construction, AISC.

b. Limitations as Seismic Space Frames. Steel moment resisting space frames, not satisfying the requirements of steel *ductile* moment resisting space frames (Steel Frame Type A, para 7-5), are limited in their use as seismic space frames by the provisions of chapter 3, paragraph 3-6 and table 3-7. The seismic coefficient K will not be less than 1.0.

c. Steel Frame Type B. This type of frame may used to resist seismic lateral forces for buildings up to 160 feet in height in Seismic Zones 1 and 2 and up to 80 feet in Seismic Zones 3 and 4. To qualify as a Steel Frame Type B, a moment resisting space frame (para 7-2*b*) will conform to the requirements of paragraph *a*, above, and the following: Each beam or girder moment connection to a column will be designed for forces resulting from the gravity loads combined with twice the design seismic moment if the connection is not designed for the full beam or girder moment capacity.

d. Steel Frame Type C. To qualify as a Steel Frame Type C, a moment resisting space frame (para 7-2*b*) will conform to the requirements of paragraph *a* above. It is permitted as a seismic space frame in Seismic Zone 1 for buildings up to 80 feet in height.

7-7. Wood frames. Wood frames will be designed using normal procedures illustrated in many easily obtainable texts and are not covered in this manual. "National Design Specification for Wood Construction" (1977 Edition and supplement), NFPA, applies.



NOTES:
WELDS UNLESS SHOWN AS FILLET WELDS ARE FULL PENETRATION BUTT WELDS. USE BACKING STRIPS OR CHIP AND USE BACKING WELDS.
THE PURPOSE OF THIS BEAM AND GIRDER CONNECTION TO THE COLUMN IS TO DEVELOP THE FULL PLASTIC CAPACITY OF BEAM AND GIRDER.
OTHER CONNECTION DETAILS WHICH ARE CAPABLE OF DEVELOPING THE PLASTIC CAPACITY OF THE CONNECTED BEAMS AND GIRDERS MAY BE USED.

Figure 7-16. Ductile Steel Frame Type A

CHAPTER 8
REINFORCED MASONRY

8-1. Purpose and scope. This chapter prescribes the criteria for design of masonry construction for buildings in seismic areas.

8-2. General. Unit-masonry shall be reinforced with deformed bars for axial, flexural, shear, and diagonal tension stresses as determined by design calculations. In addition, there are several prescribed arbitrary limitations on dimensions and reinforcement requirements; for example: (1) the minimum thickness of a wall (or partition) is governed by the type (structural role) or wall and the height between supporting diaphragms, and (2) the maximum spacing and minimum area of reinforcing bars depend upon the type of wall and the seismic zone. Additional reinforcing bars are prescribed for use around openings, at corners, anchored intersections, in wall piers, and at end of wall-panels such as at control joints. The minimum reinforcement prescribed in the manual is to provide empirical requirements relative to damage control (ductility and boundary conditions). No attempt is made to go into great detail regarding seismic load assumptions and stress distribution. These are covered elsewhere in the manual.

8-3. Definitions. Unless otherwise expressly stated, the following terms shall, for the purpose of this chapter, have the meaning indicated herein. Where terms are not defined they shall have their ordinarily accepted meanings, or such as the context may imply.

a. Reinforced Masonry. Masonry units, reinforcement, grout, and mortar combined in such a manner that the component materials act together in resisting forces, and with at least the minimum reinforcement as prescribed by this chapter:

(1) *Grouted masonry.* Multi-wythe masonry construction in which the space between wythes is solidly filled with grout.

(2) *Hollow masonry.* Single-wythe masonry construction composed of hollow units in which cells and voids containing reinforcing bars or embedded items are filled with grout as the work progresses.

(3) *Filled cell masonry.* Single-wythe masonry construction composed of hollow-units in which *all* voids are filled with grout after the wall is laid.

b. Reinforcement. Deformed reinforcing bars or joint reinforcement embedded or incased in unit-masonry in such a manner that it works with the masonry in resisting forces. Joint reinforcement is an assemblage of steel reinforcing wires designed for use in masonry bed joints, serving to distribute stresses and to tie separate wythes together.

c. Masonry Wall. A vertical, plate-like element (whose horizontal dimension exceeds five times its thickness) constructed of stone, brick, concrete masonry units, glazed structural units, or other suitable masonry materials:

(1) *Load bearing wall.* Any wall which in addition to supporting its own weight supports other loads (floors, roofs, walls, etc.).

(2) *Non-load bearing wall.* Any wall which does not intentionally support the building above it.

(3) *Shear wall.* Any wall which resists a horizontal force applied in the plane of the wall (i.e., any wall unless isolated along 3 edges).

(4) *Structural wall.* Any wall which serves in providing resistance to loads or forces other than those induced by the weight of the wall itself.

(5) *Exterior wall.* Any outer wall serving as a vertical enclosure of a building.

(6) *Partition.* Any interior wall (or vice versa).

(7) *Filler wall.* A non-bearing wall in skeleton frame construction, built between steel or concrete vertical load-carrying space frame and wholly supported at each story.

(8) *Composite wall.* A two-wythe wall in which the wythes are of different material. The wythes are so bonded as to exert a common reaction under load. GSU faced masonry and Brick/CMU grouted masonry are composite walls.

(9) *Cavity wall.* A wall built of masonry units so arranged as to provide a continuous air space within the wall (with or without insulating material) and in which both the inner and outer wythes of the wall are reinforced so as to separately resist seismic forces in proportion to their rigidities.

(10) *Veneered wall.* A masonry faced wall in which the veneer is attached to the back-up wall. It will not be considered as part of the wall in computing strength nor considered a part of the required thickness of wall.

d. Structural Members

(1) *Pilaster.* An integral portion of a wall which projects from either or both wall faces which may serve as either a vertical beam or column or both.

(2) *Column.* A compression member, vertical or nearly vertical, the width of which does not exceed

three times its thickness and the height of which exceeds four times its least lateral dimension. Any portion of a bearing wall not bonded at the sides into associated masonry shall be considered a column when its horizontal dimension does not exceed three times its thickness. The least nominal dimension of every masonry column or wall pilaster shall be not less than 12 inches. No masonry column will have an unsupported length greater than eighteen times its least nominal dimension. Refer to paragraph 8-14.

(3) *Wall-panel.* A wall segment in one plane which lies between: (1) wall ends, (2) control joints, or (3) a control joint and wall end. Each wall-panel is considered to be a separate vertical structural element.

(4) *Pier.* An upright part of a wall between (or adjacent to) openings, the width of which does not exceed five times its thickness. Design as column if width is less than three times the thickness; design as a wall if width exceeds five times the thickness. See paragraph 8-15, table 8-7, and figure 8-6.

(5) *Lintel.* A beam located over any opening in a wall to carry weight of the construction and superimposed loads above the opening.

(6) *Bond beam.* A horizontal reinforced masonry beam, serving as an integral part of the wall. Its principle purpose is to provide structural integrity and in turn crack-control. It may also serve as a chord (flange) member of a horizontal diaphragm provided reinforcement steel is made continuous for full length of the diaphragm.

(7) *Lateral support.* Members such as cross walls, columns, pilasters, buttresses, floors, roofs, or spandrel beams which have sufficient strength and stability to resist the horizontal forces transmitted to them may be considered as lateral supports.

e. Terminology

(1) *Control joint.* A continuous vertical joint in a wall designed to accommodate movements resulting from temperature and moisture changes.

(2) *Wythe.* Each continuous vertical section of a wall, one masonry unit in thickness.

(3) *Collar joint.* The continuous vertical, longitudinal joint between two wythes of masonry.

(4) *Grout.* A mixture of portland cement, aggregates, and water which is proportioned to produce pouring or pumping consistency without segregation of the constituents, serving to fill cells, voids, or collar joints in masonry walls so as to encase reinforcing and bond units together for composite action.

(5) *Mortar.* A plastic mixture of portland cement and lime (or masonry cement), fine aggregate, and water used to bond masonry.

(6) *Low-lift grouting method* contemplates grout will be poured in small increments not exce[illegible]ing 4 feet as the masonry work progresses.

(7) *High-lift grouting method* contemplates that grout will be pumped into all wall voids after the masonry units, reinforcing steel, and embedded items are built to full story height. High-lift grout is placed in one continuous pour by lifts which allow time for consolidation and loss of water, but placed at such a rate as not to form intermediate construction joints or blowouts.

f. Letter symbols are defined or illustrated where first used and arranged alphabetically in figure 8-1.

8-4. Basis of design. Previous chapters of this manual establish the basis for determining seismic forces. This chapter prescribes the criteria for the structural design of unit-masonry construction. Exterior walls, partitions, and all masonry elements will be reinforced with steel. Layout and details of construction shall be compatible with the application of the rules for modular measure. Masonry shall conform to one of the following basic types: (1) reinforced grouted masonry, (2) reinforced hollow masonry, or (3) reinforced filled-cell masonry. For any specific facility, the adoption of the type of construction, use of bases and wainscots, and selec[illegible] of materials, including contractor's options, wi[illegible] governed by manuals and guide specifications of applicable agency. For Zone 1 structures, the exception for wall reinforcement under paragraph 8-13, table 8-5, applies. Where the exception applies, masonry construction shall conform to TM 5-809-3/AFM-88-3, chapter 3 and NAVFAC DM2.6.

8-5. Design criteria. The design assumptions for reinforced unit-masonry, as regards the theory of stress distribution and analysis, will be based on the principles governing the design of reinforced concrete, except as modified hereinafter. Reinforced masonry will not be used in rigid frames. Where only intermittent cells are filled with grout, the effective area for structural sections will be governed by table 8-1 and figures 8-2 and 8-3. Several arbitrary limitations on dimensions and reinforcing are prescribed. The masonry construction must not only meet these arbitrary prescribed limits and requirements, but must also be structurally safe for the loads and forces that will be applied.

8-6. Working stresses. All reinforced masonry will be so designed and detailed that the stresses do not exceed those required by tables [illegible] and 8-3. The shear and diagonal tension stresses re-

A_g = gross area of masonry section.

E_m = modulus of elasticity of masonry.

f_a = computed axial unit stress, determined from total axial load and effective area.

F_a = axial unit stress permitted by paragraph 8-6 at the point under consideration, if member were carrying axial load only, including any increase in stress allowed.

f_b = computed flexural unit stress.

F_b = flexural unit stress permitted by paragraph 8-6, if member were carrying bending only, including any increase in stress allowed.

f'_m = ultimate compressive stress as specified in Table 8-2.

f_s = nominal working stress in vertical column reinforcement.

h = clear height in inches (paragraph 8-6, Formulas 8-1 and 8-2).

H = clear height in feet.

P = maximum concentric column axial load.

P_g = ratio of the effective cross-sectional area of reinforcement to the applicable gross area of masonry section.

t = least thickness of column in inches (paragraph 8-6, Formula 8-2).

t = nominal thickness of wall in inches (paragraph 8-6, Formula 8-1).

Δ = deflection in inches.

Figure 8-1. Symbols and Nomenclature - Reinforced Masonry

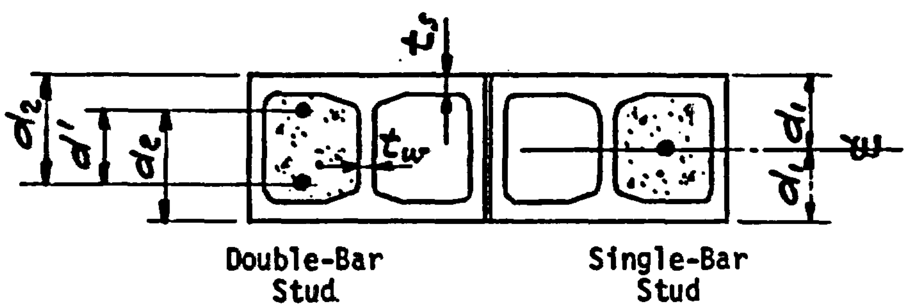
Table 8-1a. Assumed Dimensions (Inches) for Effective Area of Concrete Block (Figures 8-2 and 8-3)

Nominal Width	Design Width	Shell Width t_s	Web Width t_w	d_1	d_2	d'	x
6	5-5/8	1	1	2.81	--	--	7-1/2
8	7-5/8	1-1/4	1	3.81	5.31	3	7-1/2
10	9-5/8	1-3/8	1-1/8	4.81	7.06	4-1/2	7-1/2
12	11-5/8	1-1/2	1-1/8	5.81	8.81	6	7-1/2

Table 8-1b. Equivalent Thickness of Hollow Masonry for Computing Shear Parallel to Face (Figure 8-3(a))

Nominal Width	Spacing of Reinforcement (inches)					
	8	16	24	32	40	48
6	5.62	3.92	3.36	2.96	2.81	2.64
8	7.62	5.20	4.42	3.86	3.65	3.40
12	11.62	7.58	6.23	5.29	4.94	4.53

<u>EFFECTIVE AREA OF HOLLOW MASONRY (CMU)</u>: The working stresses to be used in the design of reinforced concrete block apply to the net section of the walls effective for resisting stress. In hollow masonry construction, the effective net section will vary and generally will be dependent upon the thickness of the face shells and cross-webs, the size of concrete-studs, and on the type of mortar bedding employed in the construction. Since contractors have the option to use standard (with plain or concave ends) or open-end two-hole concrete-masonry-units, and since exact configuration may vary between manufacturers, the precise net section will be unknown at the time of design. As a general rule, the dimensions for concrete block units may be assumed as shown in Table 8-1, and these values used in design calculations, except that the effective area shall be adjusted to reflect loss of area resulting from the use of, if any, reglets, flashing, slip-joints, and raked mortar joints.



Refer to Figure 8-3 for assumed effective area.

Figure 8-2. Assumed Dimensions for Concrete Block

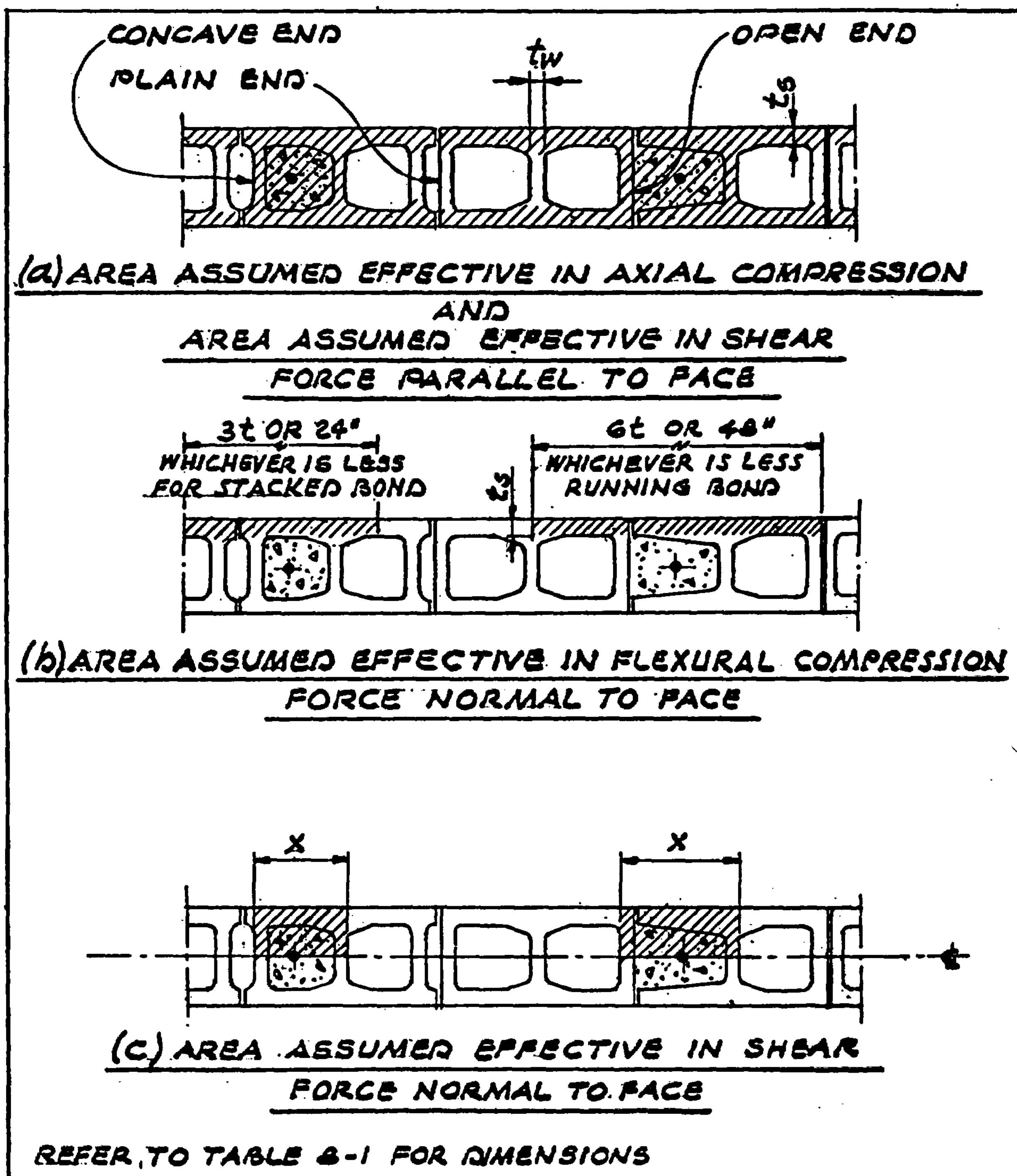


Figure 8-3. Assumed Effective Area for Hollow Masonry

Table 8-2. Basic Working Stresses for Reinforced Masonry[1]

TYPE OF STRESS	SOLID UNITS	HOLLOW UNITS[2]	SOLID & HOLLOW UNITS	
For Grades of Materials Specified[4]	f'_m = 1,500 psi[3]	f'_m = 1,350 psi[3]	f'_m	
	Building Brick: ASTM C62, Grade MW or SW Facing Brick: ASTM C216, Grade MW or SW Concrete Building Brick: ASTM C55, ASTM C145 Type N-I	Concrete Masonry Units: ASTM C90, Grade N-1 Glazed Structural Facing Units: ASTM C126 Type I Hollow Brick Unit: ASTM C652 Grade MW or SW	For materials where ultimate compressive stress (f'_m) is established by approved prism tests, but not to exceed 3,500 psi.	But Not To Exceed
COMPRESSION:				
Axial, Walls, F_a	Formula 8-1	Formula 8-1	Formula 8-1	
Axial, Columns, F_a	Formula 8-2	Formula 8-2	Formula 8-2	
Flexural, F_b	500	450	$.33f'_m$	900
SHEAR:				
No Shear Steel:[5]	40	35	$1.0\sqrt{f'_m}$	50
Full Shear Steel:[6]				
Flexural Members	115	110	$3.0\sqrt{f'_m}$	120
Shear Walls	60	55	$1.5\sqrt{f'_m}$	75
MODULUS:				
Elasticity	1,500,000	1,350,000	$1000f'_m$	3,000,000
Rigidity	600,000	540,000	$400f'_m$	1,200,000
BEARING:				
On Full Area	375	340	$.25f'_m$	900
On 1/3 or Less of Area[7]	450	400	$.30f'_m$	1,050

[1]All allowable stresses will be increased one-third when wind or seismic forces are included, provided the required section or area computed on this basis is not less than that required without wind or seismic forces.

[2]Stresses will be based on net section. Figure 8-3 applies.

[3]Where prism tests are not performed these values of f'_m may be assumed when the units comply with the applicable ASTM standards.

[4]Minimum compressive strength @ 28 days for grout and mortar will be as follows: Grout = 2000 psi, Type S mortar = 1800 psi, and Type M mortar = 2500 psi.

[5]Web reinforcement will be provided to carry the entire shear in excess of 20 psi whenever there is required negative reinforcement and for a distance of one-sixteenth the clear span beyond the point of inflection.

[6]Reinforcement must be capable of taking the entire shear.

[7]This increase will be permitted only when the edges of the loaded and unloaded area is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third but less than the full area will be interpolated between the values given.

sulting from the prescribed earthquake forces shall be increased by 50 percent (see chap 3, para 3-3(J)1h). In walls or other structural members composed of different kinds or grades of units, materials, or mortars, the maximum stress will not exceed the allowable stress for the weakest of the combinations of units, materials, and mortars of which the member is composed.

Table 8-3. Allowable Stresses for Reinforcing Bars

Type of Stress	PSI
Tensile	20,000[1]
Compression, Columns	16,000[2]
Bond—Plain Bars	60
Bond—Deformed Bars	140

[1] For deformed bars with a yield strength of 60,000 psi or more and in sizes No. 11 and smaller, use 24,000 psi.
[2] Or 40 percent of yield strength, but not to exceed 24,000 psi.

a. Allowable axial unit stresses in walls are determined by the following formula:

$$F_a = 0.20 f'_m \left[1 - \left(\frac{h}{40t}\right)^3\right] \qquad (8\text{-}1)$$

b. Allowable axial forces in columns are determined by the following formula:

$$F_a = \frac{P}{A_g} = (0.18 f'_m + 0.65 P_g f_s)\left[1 - \left(\frac{h}{40t}\right)^3\right] \qquad (8\text{-}2)$$

c. Combinations of axial and flexural stresses will satisfy the following formula:

$$\frac{f_a}{F_a} = \frac{f_b}{F_b} < 1.0 \quad \text{(or} < 1.33 \text{ when in combination with seismic or wind)} \qquad (8\text{-}3)$$

d. See Figure 8-1 for symbols and nomenclature.

8-7. General design. In calculating wall stresses, concentrated loads may be distributed over a length of wall not exceeding the center to center distance between loads. Where the concentrated loads are not distributed through a structural element, the length of wall considered shall not exceed the width of bearing plus four times the wall thickness. Concentrated loads shall not be distributed across continuous vertical joints. Due allowance will be made for the effect of eccentric loads, including additional moments caused by any end rotation of floor or roof elements framing into walls. Effective width in computing flexural stresses per reinforcing bar shall not be greater than six times the wall thickness or 48 inches for running bond or three times the wall thickness or 24 inches for stacked bond (fig 8-3(b)).

8-8. Height above grade limitation. Unit-masonry construction will not be used for shear walls where the structure exceeds 80 feet in height above the adjacent ground level. Nonstructural masonry partitions may be used with skeleton construction in structural steel or reinforced concr above the 80 feet, provided isolation compat with three times (or 3/K where K < 1.0) the floor-to-floor drift is assured by the detailing.

8-9. Vertical support. Members (girder, beams, ledgers, etc.) which provide vertical load support will be limited to non-combustible construction. The vertical support will be such that the maximum deflection of the support under all design dead and live loads will not exceed L/600 where L is the clear span of the support. To limit settlement cracking, it is essential that temporary shores be removed before erecting masonry.

8-10. Lateral support. Exterior shear walls and shear partitions shall be anchored to the structural frame or diaphragm (horizontal resisting element) by dowels, anchor bolts, or other approved methods to withstand applicable horizontal forces, normal to face, but in no case less than 200 pounds per lineal foot. Dovetail anchors are inadequate for this purpose. Nonstructural partitions should be isolated from exterior walls and shear partitions so as to prevent buttress action which would restrict shear walls from deflecting with the diaphragms. Isolated masonry partitions shall be braced to overhead construction or anchored to other isolated cross-wall assure lateral stability (refer to chap 9, para and fig 9-1). Wedges will not be used between top partition and framing.

8-11. Lintel beams. Lintels are formed by placing beam units over openings and reinforcing with a minimum of two #4 bars embedded in concrete corefill. Reinforcement shall extend 40-bar diameters or 24 inches, whichever is greater, beyond each face of opening; reinforcement shall be supported by wire chairs to insure proper coverage of steel. Steel stirrups will be provided as required. Bond beams serving as lintels shall be provided with supplemental steel as required.

8-12. Bond beams. Reinforcement bars in bond beams will be lapped 40 diameters or 24 inches, whichever is greater, at splices, at intersections, and at corners. Bar splices will be staggered. Bond beams will be provided at top of masonry foundation wall stems, below and at top of openings or immediately above lintels, at floor and roof levels, and at top of parapet walls. Intermediate bond beams will be provided as required to conform to the maximum spacing of horizontal bars (para 8-13*b*, table 8-5). However, whenever the height is not a multiple of this normal spacing, the spacing m increased up to a maximum of 24 inches prov

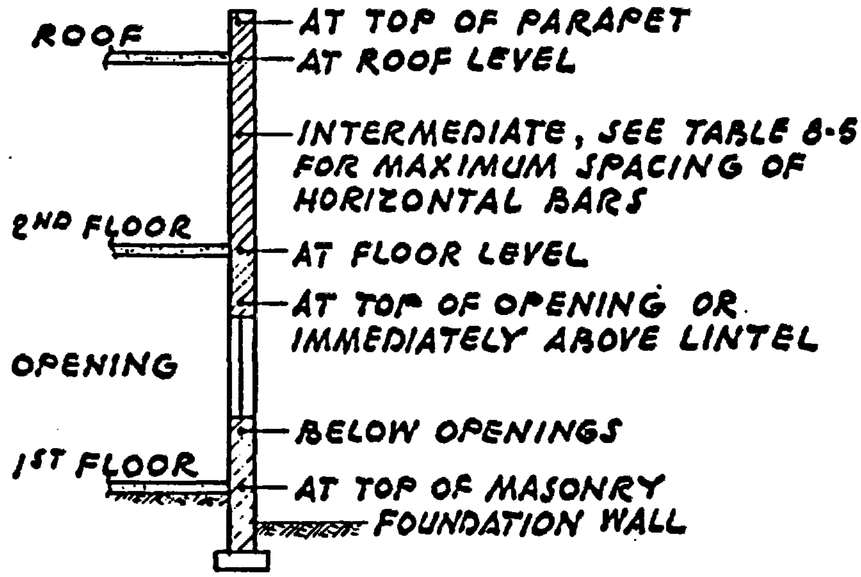


Figure 8-4. Location of Bond Beams

the bond beams are supplemented with joint reinforcement. One line of joint reinforcement will be provided for each 8-inch increase in the spacing. No additional bond beam will be required between window openings which do not exceed 6 feet in height, provided the prescribed supplemental joint reinforcement is installed. To facilitate placement of steel or concrete corefill, the top bond beam for filler walls or partitions may be placed in next to top course. The area of bond beam reinforcement shall be included as part of the minimum horizontal steel. See figure 8-4.

8-13. Walls and partitions. Masonry walls and partitions shall be designed for applicable vertical loads and horizontal forces, both parallel and normal to face, with due allowance for the effect of any eccentric loadings. Since distribution of lateral forces to any wall-panel depends upon the relative stiffness of the various vertical resisting elements at the particular level, the location of control-joints must be established before distribution of the lateral forces is made. For more complete discussion of lateral force distribution refer to chapter 4, paragraph 4-4, and chapter 6, paragraph 6-2. The resulting stresses will comply with the requirements of paragraph 8-6. In addition, there are certain prescribed arbitrary limitations on wall dimensions, minimum reinforcement, and maximum bar spacings.

a. Height and Thickness Limitations. The minimum nominal thickness of a wall is controlled by the type (structural role) of the wall and the height and width between supports. Table 8-4 applies.

b. Minimum Reinforcement. Unit-masonry needs to be reinforced not only for structural strength but to provide ductile properties and to hold it together in the event of severe seismic disturbance. All walls and partitions will be reinforced as required by structural calculations, but in no case, less than the minimum area of steel and the maximum spacing of bars prescribed below. The minimum reinforcement and the maximum spacing of bars is controlled by the type of wall and the seismic zone. Table 8-5 applies. Only reinforcement which is continuous in any wall-panel will be considered in computing the minimum area of reinforcement. Joint-reinforcement used for crack-control or mechanical bonding may be considered as part of the total minimum horizontal reinforcement, but will not be used to resist computed stresses. Further, additional bars will be provided around openings, at corners, anchored intersections, in wall piers, and at ends of wall-panels as prescribed elsewhere in this chapter. Vertical bars in walls will be lapped spliced 40 diameters or 24 inches minimum.

Table 8-4. Maximum Unsupported Wall Height or Lengt

Type of wall	Nominal wall thickness (inches)	Max. height or length between diaphragms or supports (feet)
Structural (load-bearing or shear)	6	12
	8	16
	10	20
	12	24
	14	28
	16	32
Nonstructural	4*	10*
	6	18
	8	24
	10	30
	12	36
	14	36
	16	36

*4-inch walls in Zone 1 only in buildings not exceeding three stories.

8-14. Columns and pilasters. Masonry umns and pilasters (fig 8-5) will be construct reinforced masonry as prescribed by this chapter, and will be designed to withstand all horizontal and vertical loads. Masonry columns or pilasters will not be used to qualify a structure for a complete vertical load-carrying space frame so as to reduce the factor "K" below 1.33 of a box system. Masonry columns will not be used in rigid frame construction.

a. Limiting Dimensions. The least nominal dimension of every masonry column or wall pilaster will be not less than 12 inches. No masonry column or pilaster will have an unsupported length greater than 18 times its least nominal dimension. Table 8-6 applies (also, see para 8-3*d* and table 8-7).

b. Allowable Loads. The maximum allowable axial load on columns and pilasters will be governed by paragraph 8-6 (formula 8-2).

c. Vertical Reinforcement. Vertical reinforcement will be neither less than $0.005A_g$ nor more than $0.04A_g$, where A_g is gross area of column. Not less than four #4 bars will be used. Bars will be lapped 30 diameters.

d. Lateral Ties. Hoop ties of not less than #2 bars for #7 or smaller vertical reinforcement and (s for larger reinforcement will be spaced apa t

Table 8-5. Minimum Wall Reinforcement

	Total Minimum reinforcement (percent)[1,2]			Maximum spacing of bars (inches)					
				Vertical bars			Horizontal bars		
	Seismic Zone			Seismic Zone			Seismic Zone		
	4&3	2	1[3]	4&3	2	1[3]	4&3	2	1[3]
Structural	0.20	0.20	0.15	24	36	60	48	60	72
Nonstructural	0.15	0.15	0.15	48	60	72	84	84	96

NOTES
[1]The total minimum reinforcement is the sum of the vertical and horizontal reinforcement; not less than 1/3 of the prescribed total minimum reinforcement will be used in either direction.
[2]The percentage of area reinforcement is based on gross area of wall (nominal dimensions).
[3]*Exception:* In Seismic Zone 1, one story structures with eave heights not exceeding 14 feet; and two and three story structures with story heights not exceeding 12 feet may be reinforced or partially reinforced masonry. These structures must be capable of resisting seismic zone 1 loads but will be designed by the usual non-seismic criteria. (Partially reinforced masonry shall be designed as unreinforced masonry except that reinforcement is provided in some areas to resist flexural tension stresses. The maximum spacing of vertical reinforcement shall be 8 feet. Vertical reinforcement shall be provided at each side of each opening and each corner of all walls. Horizontal reinforcement shall be provided at top of footings, at bottom and top of openings, at roof and floor levels, and at top of parapet walls.)

over 16 bar diameters, 48 tie diameters, or the least nominal dimension of the column. Lateral ties will be in contact with the vertical steel and not in the horizontal bed joints. Lateral ties shall be placed not less than 1-1/2 inches nor more than 3 inches from the top of column. Additional ties of three #3 bars shall be placed within the top 5 inches of column.

8-15. Wall piers. Masonry wall piers will be designed to withstand all horizontal and vertical loads. Every pier or wall section whose height exceeds four times its thickness and whose width is less than three times its thickness will be designed and constructed as required for columns. Every pier or wall section whose width is between three and five times its thickness will have all horizontal steel in the form of ties. Table 8-7 and figure 8-6 apply.

8-16. Wall openings. Since the area around wall openings is vulnerable to failure, supplemental reinforcement around the perimeter of openings is prescribed herein. For purpose of this paragraph, the term "jamb bars" shall mean bars of the same size, number, extent, and anchorage as the typical vertical stud reinforcement in that wall, and in no case less than one bar, #4 or larger. Refer to figure 8-7.

a. Case I. Provide jamb bars on each side of opening *and* at least one bar, #4 or larger, at top and bottom of opening. The lintel bars above the opening may serve as the top horizontal bar and a bond beam bar at the bottom of the opening may serve as the bottom horizontal bar. Case I applies to: (1) all openings in nonstructural partitions over 100 square inches, and (2) any opening in structural partitions or exterior walls which is 2 feet or less both ways but over 100 square inches.

b. Case II. The perimeter reinforcement will be the same as in Case I *plus* additional reinforcement as follows: provide at least one bar, #4 or larger, on all four sides of the opening in addition to required bars in Case I and shall extend not less than 40 bar diameters or 24 inches, whichever is larger, beyond corners of the opening. Case II applies to exterior walls and structural partitions for any opening which exceeds 2 feet but not over 4 feet in any direction.

c. Case III. The perimeter reinforcement will be the same as in Case II, except that vertical jamb bars will be provided in lieu of the shorter vertical bars. Case III applies to any opening which exceeds 4 feet in either direction in exterior walls or structural partitions.

8-17. Stacked bond. Since a running bond pattern is the strongest and most economical, the criteria in this manual are based upon each wythe of

Table 8-6. Column or Pilaster Height Limitation

Least Dimension (inches)	12	14	16	20	24
Maximum Height (feet)	18	21	24	30	36

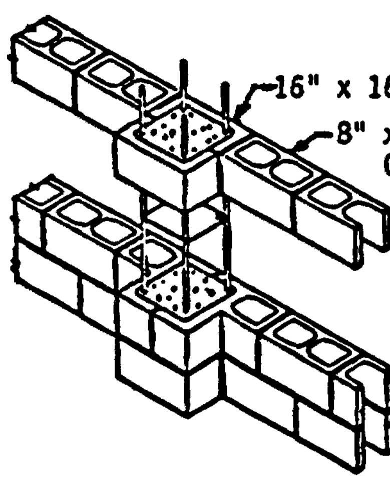


Figure 8-5. Hollow Unit Masonry Pilaster

Table 8-7. Dimensions of Wall Piers (Inches)

Nominal Wall Thickness (inches)	Design as a Column If:		Design as a Pier If W Equals	Design as a Wall If W Exceeds
	W less than	h greater than		
6	24*	24	24 - 32	32
8	24*	32	24 - 40	40
10	32*	40	32 - 48	48
12	40	48	40 - 64	64
16	48	61	48 - 80	80
Design Criteria	Paragraph 8-14		Paragraph 8-15	Paragraph 8-13
	For additional reinforcement around openings, see paragraph 8-16			

*Requires pilaster

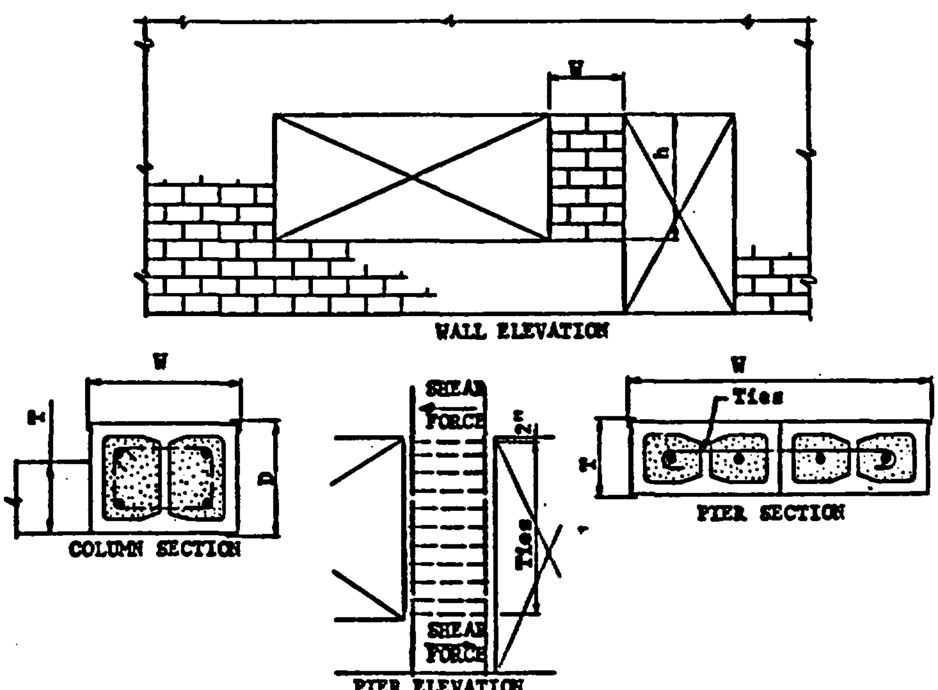
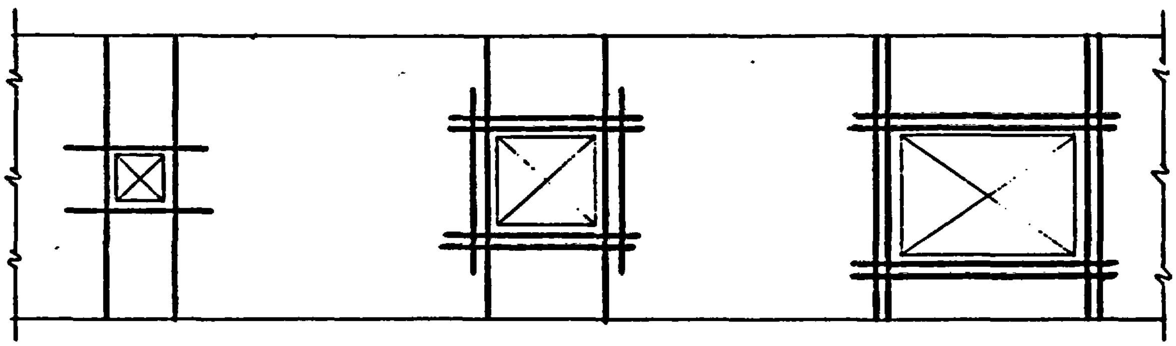


Figure 8-6. Dimensional Limitations for Masonry Piers and Walls



Refer to paragraph 8-16 for application of Cases I, II, and III.

Figure 8-7. Reinforcement Around Wall Openings masonry walls being constructed in a running bond pattern. Use of stacked bond pattern will be restricted to reinforced walls essential to the architectural treatment. Filled cell masonry or grouted masonry shall be used. For filled cell masonry, open end blocks shall be used and so arranged that closed ends are not abutting.

8-18. Cavity walls. This form of construction is commonly used where resistance to rain penetration is desired and where thermal insulation may be provided. The two wythes of the wall forming the cavity must be separately reinforced and thus designed as independent structural walls. There is no limitation on the width of the cavity. The wall thickness and heights must comply with table 8-4. If the exterior wythe is tied to the reinforced inner wythe but is nonbearing and isolated on three sides, the exterior wythe may be unreinforced, in which case this construction may be considered as an anchored veneer and must comply with requirement for anchored veneer.

EXCEPTION: Seismic Zone 1, see table 8-5 exceptions, cavity walls may be designed in accordance with TM-5-809-3, AFM 88-3, chapter 3 and NAVFAC DM-2.6.

8-19. Veneered wall. There are two methods for attaching veneer to a backup structural wall (see fig 8-8).

a. Anchored veneer is a masonry facing secured by joint reinforcement or equivalent mechanical tie attached to the backup. All required load carrying capacity (both vertical and lateral) shall be provided by the structural backup wall. The veneer shall be nonbearing and isolated on three edges to preclude it from resisting any load other than its own weight and in no case shall it be considered part of the wall in computing required thickness of a masonry wall. The veneer shall be not less than 1-1/2 inches nor more than 5 inches thick. The veneer will be tied to the structural wall with 3/16 inch round corrosion resisting metal ties or joint reinforcement capable of resisting in tension or compression, the wind load or two times the weight of veneer, whichever governs. Maximum spacing of ties is 16 inches and a tie must be provided for each two square feet of wall area. Adjustable ties are not permitted. The maximum space between the veneer and the backing shall not exceed 2 inches unless spot mortar bedding is provided to stiffen the ties. A noncombustible, noncorrosive horizontal structural framing shall be provided for vertical support of the veneer. The maximum vertical distance between horizontal supports shall not exceed 25 feet above the adjacent ground and 12 feet maximum spacing above the 25 feet height. The deflection of a supporting lintel will be limited to L/600.

b. Adhered veneer is masonry veneer attached to the backing with minimum 3/8 inch to maximum 3/4 inch mortar or with approved thin set latex Portland cement mortar. The bond of the mortar to the supporting element shall be capable of withstanding a shear stress of 50 p.s.i. Maximum thickness of the veneer shall be limited to 1 inch. Since adhered veneer is supported through adhesion to the mortar applied over a backup, consideration shall be given for differential movement of supports including that caused by temperature, shrinkage, creep, and deflection. A horizontal expansion joint in the veneer is recommended at each floor level to prevent spalling. Vertical control joints should be provided in the veneer at each control joint in the backup.

8-20. Three basic types of reinforced masonry walls. *a. Reinforced grouted masonry* is that type of construction made with two wythes of masonry units in which the collar joint between is reinforced and filled solidly with concrete grout. The grout may be placed as the work progresses or after the masonry units are laid. Collar joints will be reinforced with deformed bars, both vertical and horizontal. Reinforcement and embedded items such as structural connections and electrical conduit shall be positioned so as to allow proper placement of grout. All units will be laid in running bond with full shoved head and bed mortar joints. Masonry headers will not project into grout spaces. Clipped-brick headers will be used where the appearance of masonry headers is required. See figure 8-9.

(1) *High-lift grouting* procedures contemplate that: first, both vertical and horizontal bars are erected; then, the masonry units are laid, one wythe of masonry on each side of the reinforcement, with space between for grout; finally, after the masonry is built a full story height, the collar joint is filled solidly with concrete grout. As the work progresses, both wythes shall be kept approximately at the same height to accommodate the wall ties (or ladder bars) spaced not to exceed 24 inches horizontally and 16 inches vertically to resist the hydrostatic pressures of the fluid grout. These ties shall be laid in the mortar bed and all ties shall be in the same line vertically in order to facilitate the vibrating of the grout pours. Width of the grout space shall be not less than 3-1/2 inches and the wall shall be constructed so as to preserve an unobstructed vertical alignment of the grout space. Cleanout openings shall be provided at the bottom of each pour. The openings shall be of sufficient size and location to

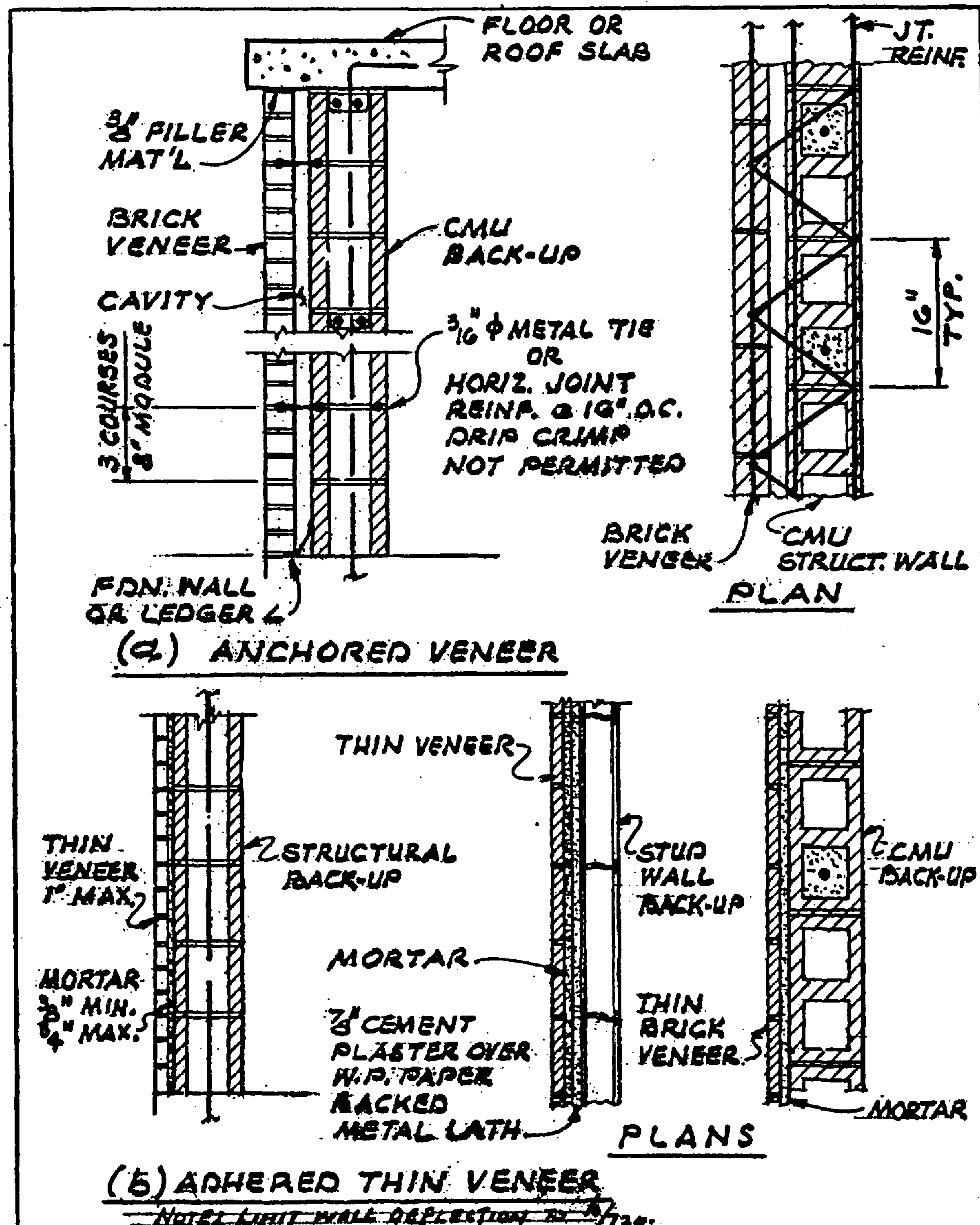


Figure 8-8. Veneered Walls

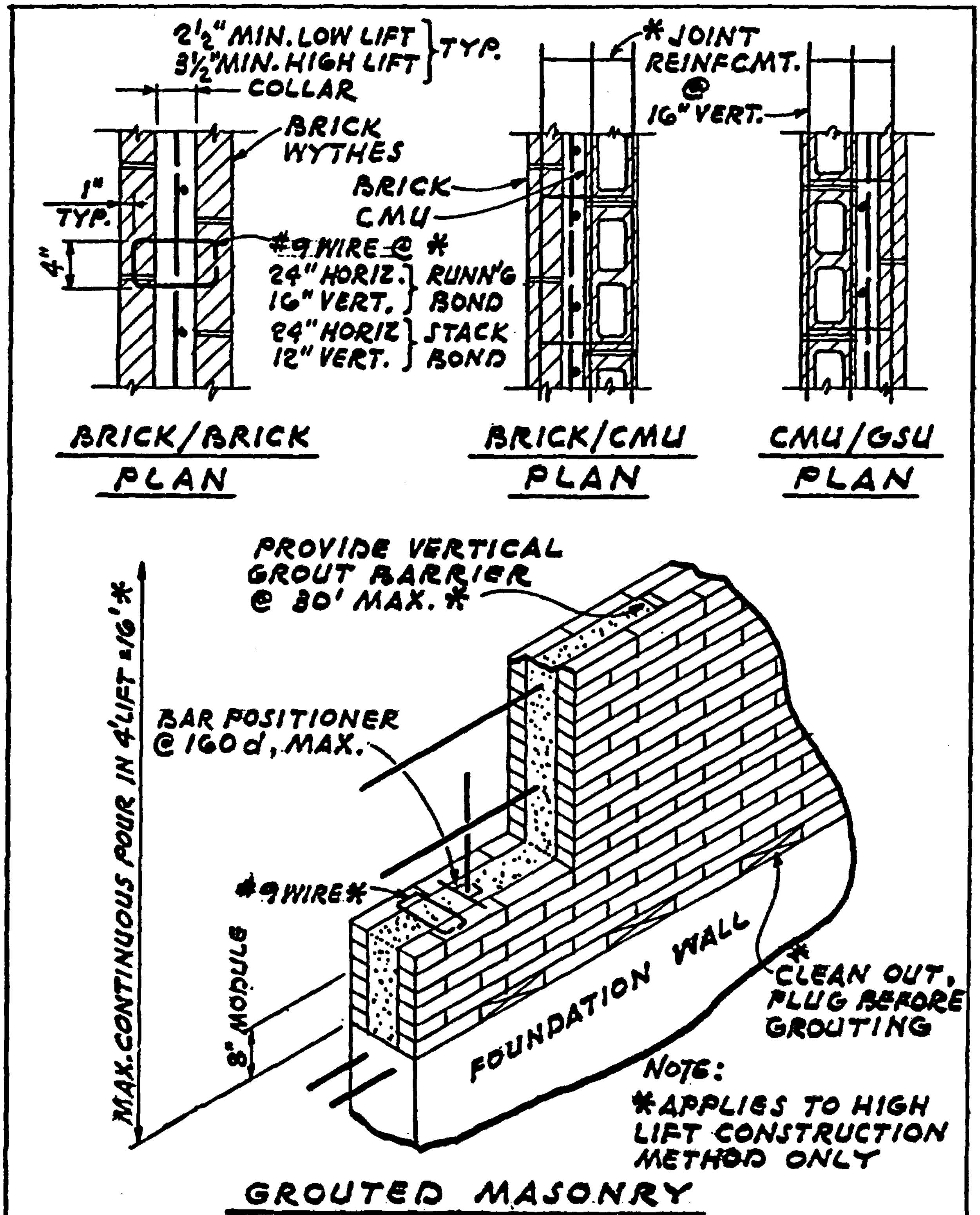


Figure 8-9. Reinforced Masonry Walls

allow flushing away of mortar droppings and debris. All mortar droppings and overhangs shall be removed from the foundation or bearing surface and reinforcing. A sand bed or plastic film will prevent mortar droppings from bonding to the foundation wall. Dislodge any hardened mortar from the collar joint wall surfaces and reinforcing with a pole or rod and remove the mortar debris prior to cleanup and grouting. All cleanout closures, reinforcing, bolts, and embedded connection items shall be in position before grouting is started. Grout shall be handled from the mixer to the point of deposit in the grout space as rapidly as practical by pumping and placing methods which will prevent segregation of the mix and cause a minimum of grout splatter on reinforcing and masonry unit surfaces not being immediately encased in the grout lift. Use of the high-lift grouting methods should be restricted to walls where wall openings, arrangement of piers, special reinforcing details, or embedded items do not prevent the free flow of grout or inhibit the use of mechanical vibration to properly consolidate the grout. A grout admixture is recommended to reduce early water loss to the masonry units and to produce a slight expansion sufficient to offset initial shrinkage and promote bonding of the grout to the interior surface of the units.

(2) *Low-lift grouting* procedures contemplate that: first, the vertical bars are erected; then, the horizontal bars are placed and grouted in as laying of the masonry work progresses. The contact surface of all foundations and floors that is to receive masonry work shall be cleaned and roughened to insure a good bond between the grout fill and the concrete surfaces. Width of collar joints shall be such as to provide at least 1/2 inch grout coverage around all reinforcement bars.

b. Reinforced hollow masonry is that type of construction made with a single wythe of hollow masonry units (concrete or clay blocks), reinforced vertically and horizontally with steel bars, and cores and voids containing reinforcing bars or embedded items are filled with grout as the work progresses. Construction procedures contemplate that the vertical bars are erected first; then, the horizontal bars and joint reinforcement, if required, are placed and grouted in as laying of the hollow masonry work progresses. See figure 8-10.

c. Reinforced filled-cell masonry is that type of construction made with a single wythe of hollow masonry units, reinforced vertically and horizontally with deformed steel bars, and *all* cores and voids are filled solidly with grout after the wall is laid. Construction procedures contemplate that, first, the hollow masonry units are laid to full height of th wall with horizontal bars and joint reinforcement being placed as the masonry work progresses; the vertical bars may be either erected first or dropped into position after the wall is erected. Finally, all cores and voids are grouted solidly by the high-lift grouting method. Use of open end units is preferred and bond-beam units are required at all horizontal bar locations. Both horizontal and vertical reinforcement shall be held in position by wire ties or spacing devices near each end and at intervals not exceeding 160-bar diameters. The contact surface of all foundations and floors that are to receive masonry work shall be cleaned and roughened before start of laying. It shall be protected during construction to insure a good bond between the grout fill and concrete surfaces. Cleanout openings shall be provided through block faces at the bottom of each pour, of sufficient size and location to allow flushing away of mortar droppings and debris. After laying of masonry units is completed, the cells cleaned, reinforcing positioned, inspection completed, and cleanouts closed, the high-lift grout shall be placed in one continuous pour by lifts which allow time for consolidation and loss of water, but placed at such a rate as not to form intermediate construction joints or blowouts. The maximum height of any pour shall be limited to 12 feet for 8-inch walls and 16 feet for 12-inch walls. Low-lift grouting procedures may also be used for filled cell construction. See figure 8-11.

8-21. Control joints (crack control). Cracking of walls constructed with concrete-masonry-units is caused by the development of tensile stresses within the wall assembly which exceed the tensile strength of the materials comprising the assembly. Generally it is due to tensile stresses which develop when wall movements accompanying temperature and moisture change as restrained by other elements, or when concrete masonry places restraint on the movements of adjoining elements. Moisture loss depends on the shrinkage potential of the masonry units and the drying conditions at the building site, expressed in terms of relative humidity. Major methods employed to control cracking in masonry structures are (1) materials specifications to limit the drying-shrinkage potential, (2) reinforcement to increase crack resistance, and (3) control joints to accommodate movement. Any crack control measure taken must be compatible with the structural design for seismic forces. Control joints provide a complete separation of the masonry. Hence, location of control joints fixes the length of wall-panels and, in turn, the rigidity of the walls, the distribution of seismic forces and the resulting unit stresses. Therefore, adding, eliminating or relocating control joints will not be permitted once the structural design is complete. Control joints shall never be assumed to transfer bending moments or diagonal tension across the joint. Joint reinforcement and bars in nonstructural bond beams will be terminated at control joints; deformed bars in structural bond beams will be made continuous for length of the diaphragm. Using quality controlled concrete-masonry-units and the prescribed minimum reinforcement of seismic design, cracking is not normally a problem when the maximum horizontal spacing of control joints is limited to four times the diaphragm-to-diaphragm height or 100 feet on center, whichever is less. See figure 8-12.

8-22. Connections to other elements. The use of joints and connections for the transmission of shears, axial loads, moments, and torsions from diaphragms to walls and from walls to sub-structure is inherent in seismic design. Great care must be taken to properly design connections between the vertical resisting elements (shear wall-panels) and the horizontal resisting elements (floor and roof diaphragms) so as to make such walls an integral part of the structural system. Positive means will be provided for transferring shear from the plane of the diaphragm into the shear wall-panels into the diaphragms. In designing connections or ties, it is necessary to carry out the forces and their stress paths (according to relative rigidity) and also to make each and every connection along each path adequate and consistent with the basic assumptions and distribution of forces. Because joints and connections directly affect the integrity of the structure, their design and fabrication must be adequate for the functions intended. In designing and detailing, it is well to keep in mind that the lateral forces are not static, as assumed for convenience, but dynamic and to a great extent unpredictable.

a. Forces to be considered in the design of joints and connections are gravity loads; temporary erection loads differential settlements; horizontal loads normal to wall; horizontal forces parallel to wall; and creep, shrinkage, and thermal forces—separately or combined as applicable. Bond beams acting as flange (chord) for horizontal diaphragms will require reinforcement to be continuous at dummy control joints for tensile and compressive chord stresses induced by the diaphragm beam action, and the marginal connections must be capable of resisting the flexural and shear stresses developed.

b. Joints and connections may be made by welding steel reinforcement to structural steel members, by bolting, by dowels, by transfer of tensile or compressive stresses by bond of reinforcing bars, or by use of key-type devices. The transfer of shear may be accomplished by using reinforcing steel extended as dowels coupled with cast-in-place concrete placed between roughened concrete interfaces, mechanical devices such as embedded plates or shapes. The entire shear should be considered as transferred through one type of device, even though a combination of devices may be available at the joint or support being considered. Maximum spacing of dowels or bolts will not exceed 4 feet. All significant combinations of loadings should be considered, and the joints and connections should be designed for forces consistent with all possible combinations of loadings. Details of the connections shall admit to a rational analysis in accordance with well-established principles of mechanics.

c. The strength of connections, as a general rule, should be sufficient to develop the useful strength of the structural elements connected, regardless of calculated stress. The design forces for joints and connections between lateral force resisting elements will be at least 2.0 times the calculated shear when using the prescribed lateral loads, except that the connection need not be required to develop forces greater than the ultimate capacity of the connected elements, and in no case less than 200 pounds per linear foot. The shear on every bolt shall not exceed the values given in table 8-8.

d. Cautionary Notes for Designers and Detailers. Avoid connection and joint details which would result in stress concentrations that might result in spalling or splitting of face shells at contact surfaces. Liberal chamfers, adequate reinforcement, and cushioning materials are a few means by which stress concentrations may be avoided or provided for. Avoid direct bearing of heavy concentrated loads on face shell of concrete masonry units. Avoid welding to any embedded metal items which might cause damage to the adjacent masonry by spalling, in particular where the expansion of the heated metal is restrained by masonry. All bolts and dowels which are embedded in masonry will be grouted solidly in place with not less than 1 inch of grout between the bolt or dowel and the masonry. At tops of piers and columns, vertical bolts will be set inside the horizontal ties.

8-23. Fire walls. A fire wall is a fire-resistive barrier which must be able to withstand the temperature of uncontrolled fires without disintegration, prevent passage of fire from either side to the other

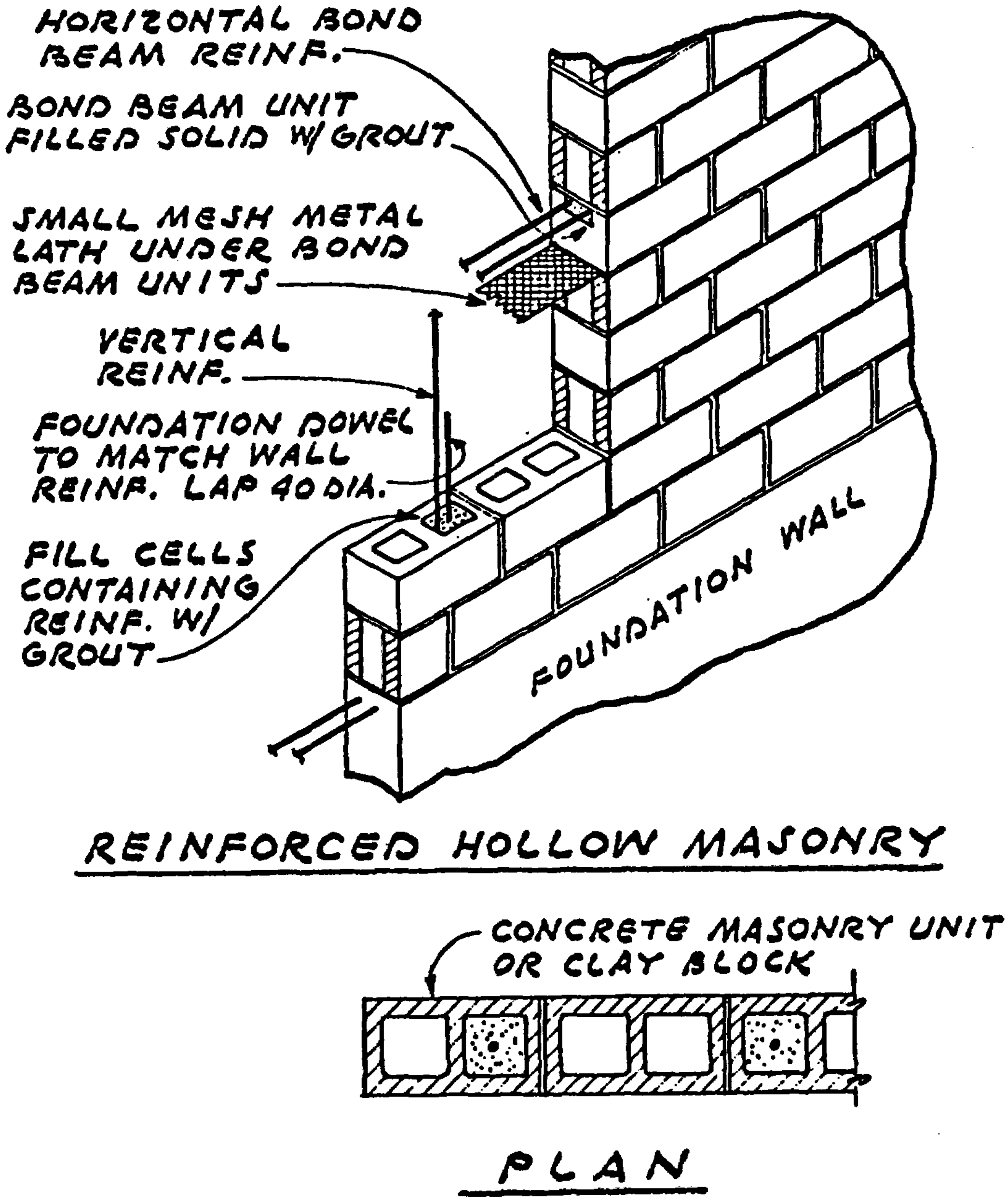


Figure 8-10. Reinforced Hollow Masonry

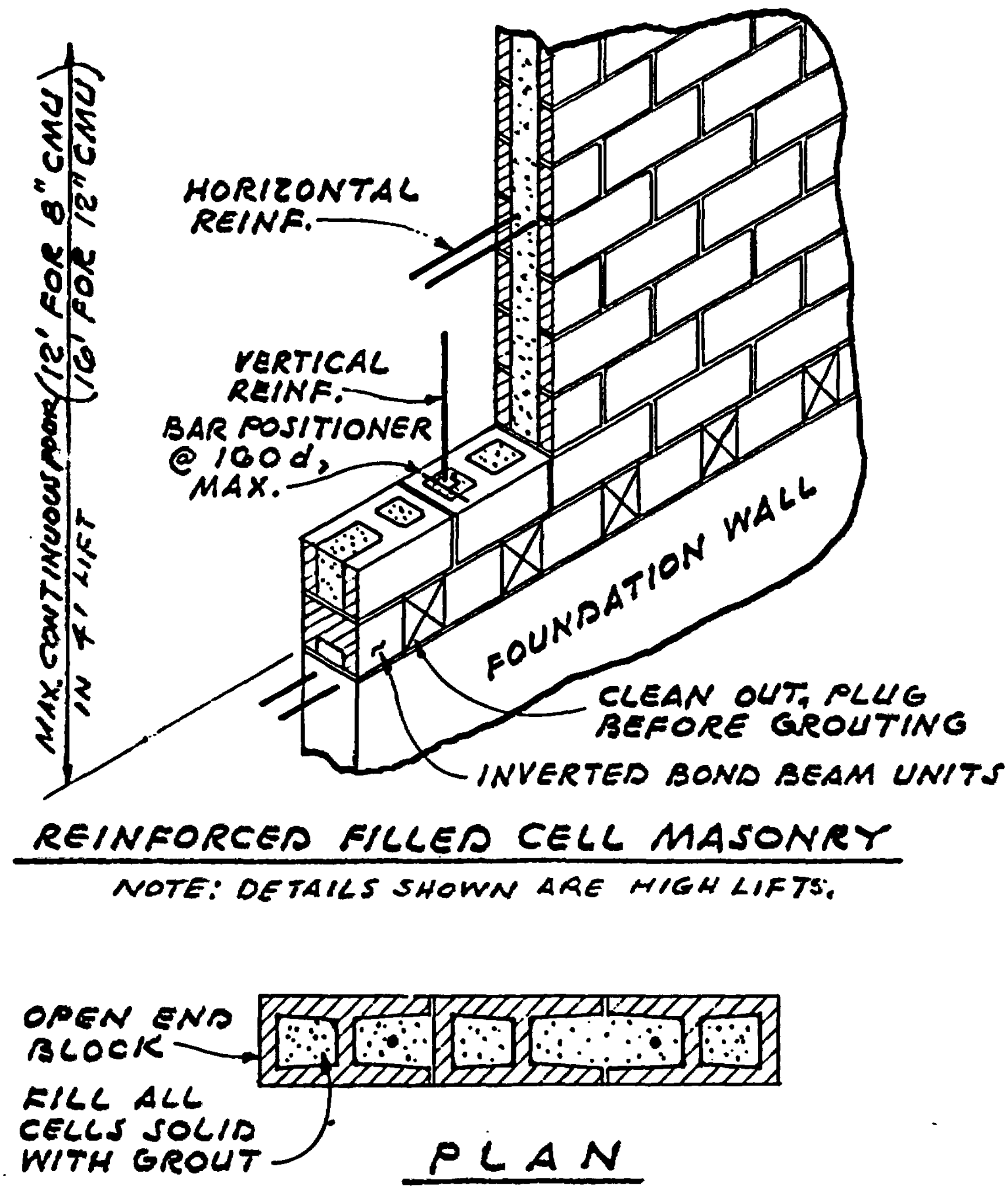


Figure 8-11. Reinforced Filled Cell Masonry

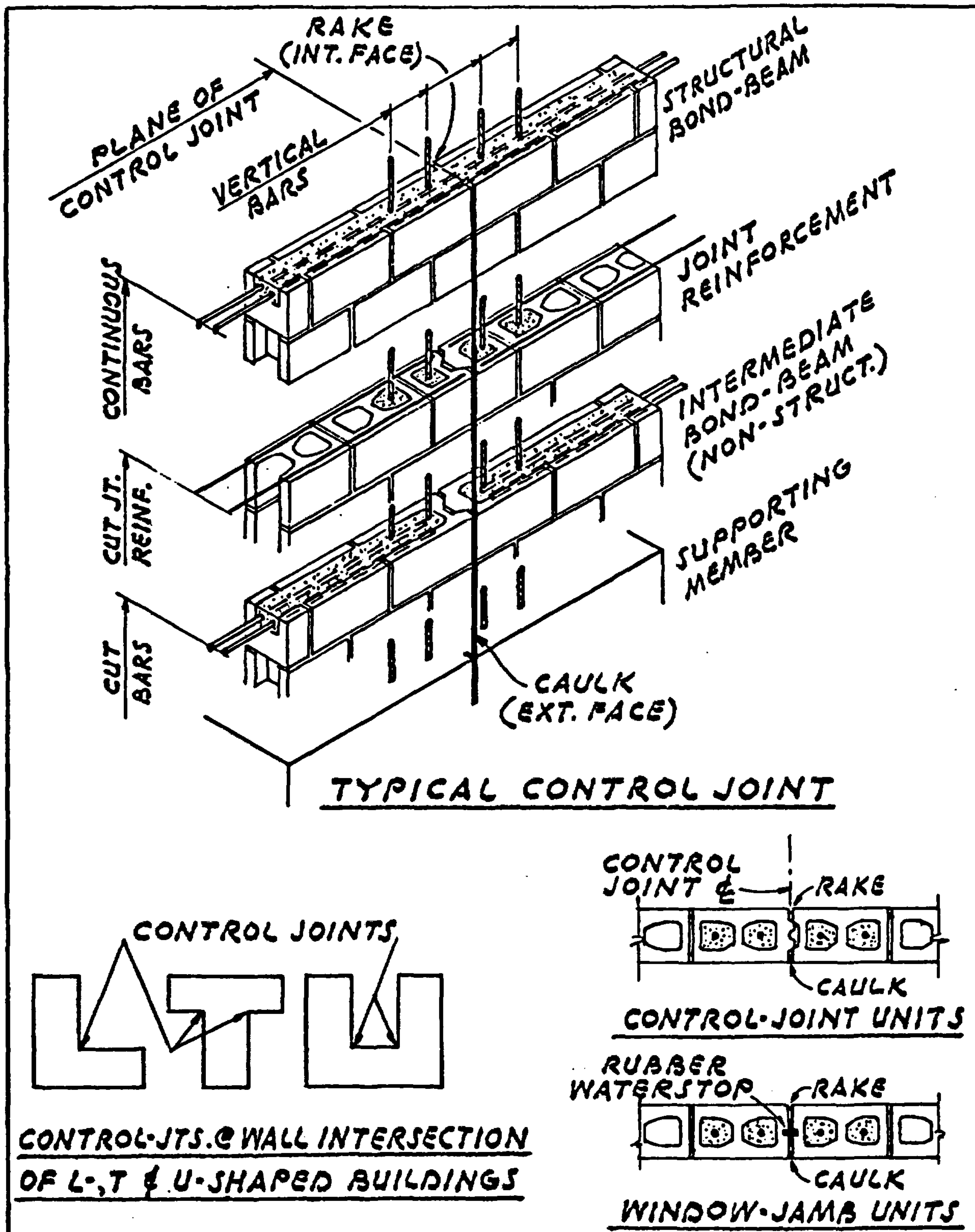


Figure 8-12. Control Joints

Table 8-8. Allowable Shear on Bolts and Dowels

Diameter (inches)	Minimum Embedment and Spacing (inches)**	Shear (pounds)	Minimum Edge Distance in Loaded Direction (inches)
1/2	4	350	3
5/8	4	500	3
3/4	5	750	4
7/8	6	1,000	4
1	7	1,250*	5
1-1/8	8	1,500*	5

For load applied at the top of and parallel to wall, the bolt values may be increased 50 percent when vertical bolts are set between horizontal bond-beam reinforcement.
Allowable shear may be increased 1/3 when wind or seismic forces are included.
*Permitted only with not less than 2,500 psi units.
**An additional 2 inches of embedment will be provided for anchor bolts at top of columns.

side, confine fire from sweeping over or around either end (by use of parapet at roof line, and wing walls or fire wall returns at exterior walls), have insulating qualities to maintain low temperature on the unexposed face of the wall, and remain standing even when a portion of the building on either side collapses. Stability is one of the essential properties of a fire wall. Such a wall must remain standing during a fire even when the building framing on one side collapses. This stability requirement has led to several little-appreciated design problems in location of expansion (seismic) joints and in selecting or adapting a seismic structural system which is compatible with this fire wall requirement. A "Fire Cut-Off Partition" is a fire-resistive barrier used to delay the spread of a fire; but, unlike a fire wall, it is not required to remain standing should a portion of the building collapse. The most commonly used types of fire walls are described below (other types may be used, provided they conform to the principles in the foregoing text). (Refer to fig 8-13.)

a. Double fire wall is a very reliable type of fire separation. The separate walls are laterally supported by their respective building structural system, and each may be part of a seismic structural system. In case of masonry, it may be used as two shear walls back-to-back. If there is an uncontrolled fire on either side of the double wall, the building frame will collapse and pull one wall with it. The other wall, being supported by the framing on the side away from the fire, will remain in place. A double wall having two 3-hour one-way walls may be considered as a 4-hour wall. The double wall serves as an expansion joint in the building. The width of the gap between is based on requirements for seismic joints.

b. Free-standing fire wall, as an alternative to a double fire wall, is entirely self-supporting without any structural tie to adjacent framing. For stability against horizontal forces, it must rely on its own strength as a cantilever from the base. Horizontal forces may be caused by wind, earthquake, or by the pull of flashing as the burning portion of the building collapses. Lateral strength of the wall shall be obtained by providing reinforcing steel in the wall and by adding reinforced pilasters, if necessary. A double seismic joint is required and each portion of the building, adjacent to fire wall, will be designed as an independent structure.

c. One-way fire wall meets all requirements of a regular fire wall except that it is limited to remain standing when the fire exposure is from one (preselected) side. Therefore, it is useful only to isolate a hazardous area from an ordinary or light hazardous occupancy.

8-24. Weatherproofing. Each job requires a separate decision as to the requirements for weatherproofing, damp-proofing, thermal insulation, and vapor control. Manuals and guide specifications of applicable agency apply.

8-25. Surface bonding of concrete masonry units. This method of construction is not permitted in Seismic Zones 2, 3, and 4. Use in Zone 1 is restricted to design agency approval.

8-26. Drawings. The locations of control joints, and the identification of structural and nonstruc-

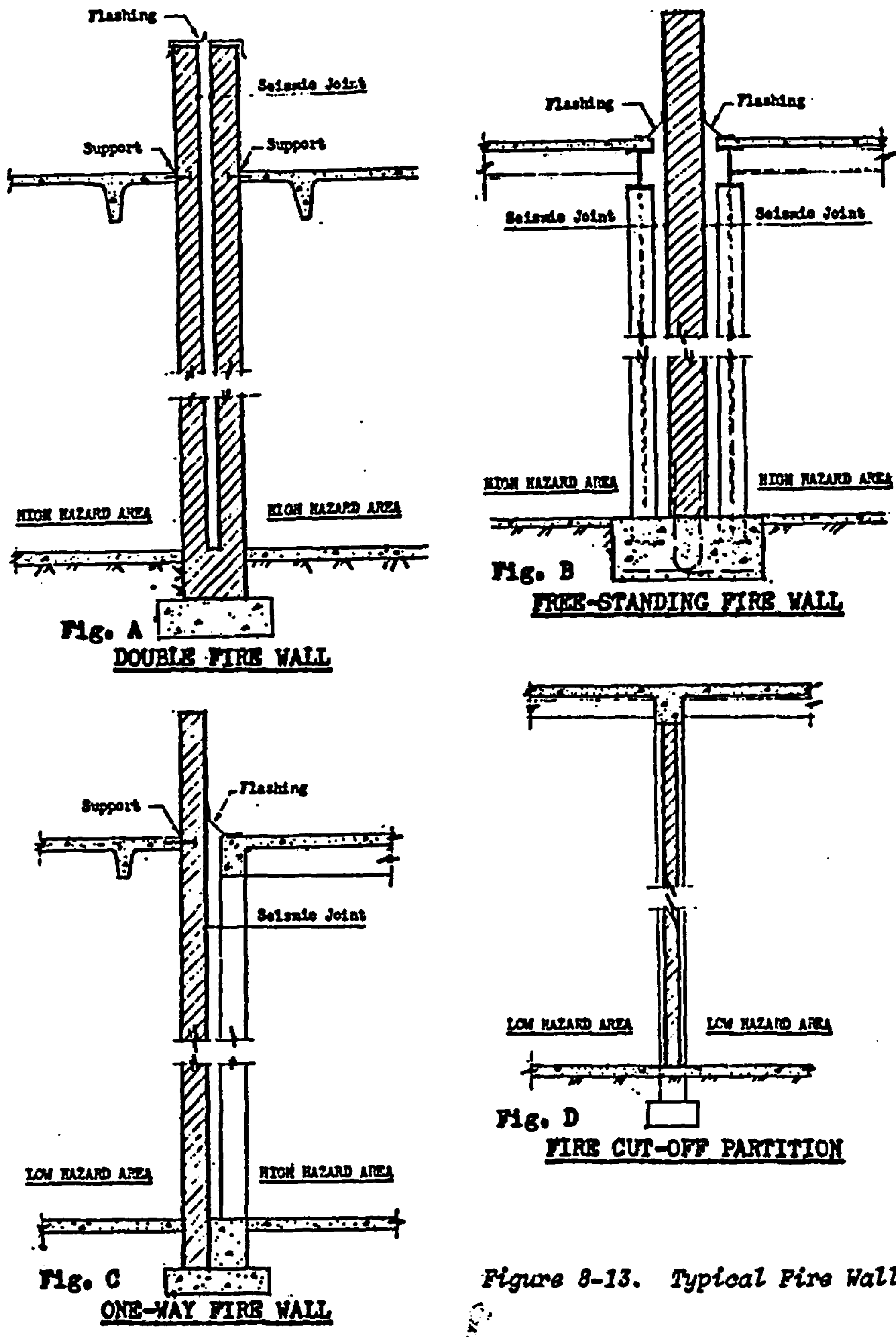


Figure 8-13. Typical Fire Walls

tural walls and partitions for all masonry construction will be shown on preliminary and contract drawings. On contract drawings, show complete details for masonry, reinrcement, and connections to other elements. Detailing procedures outlined in ACI-315, "Manual of Standard Practice for Detailing Reinforced Concrete" are generally applicable to reinforced masonry.

8-27. Overseas construction. In overseas construction, where local materials or grades other than those herein are used, the working stresses, details, and other requirements of this chapter will be modified as required because of the characteristics of the materials.

8-28. Additional details. See figures 8-14 through 8-17, and tables 8-9 and 8-10.

8-29. References.

a. Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, California, 90057, "Masonry Design Manual," (1969).

b. National Concrete Masonry Association, 6845 Elm Street, McLean, Virginia, 22101, "Technical Notes (TEK) Information Series (1970).

c. Brick Institute of America, 1750 Old Meadow Road, McLean, Virginia, 22101, "Technical Notes on Brick and Tile Construction."

d. American Concrete Institute, P.O. Box 4754, Redford Station, Detroit, Michigan, 48219, Report of ACI Committee 531; "Concrete Masonry Structures-Design and Construction," (1970). (Also refer to ACI Journal papers No. 75-42, August 1978 and No. 75-50, September 1978.)

e. Amrhein, James E., "Reinforced Masonry Engineering Handbook," 3rd edition, (1978); Masonry Institute of America, Los Angeles, California.

f. Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois, 60076, "Concrete Masonry Handbook for Architects, Engineers, Builders," (1976).

g. Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, California, 90057, "1977 Masonry Codes and Specifications."

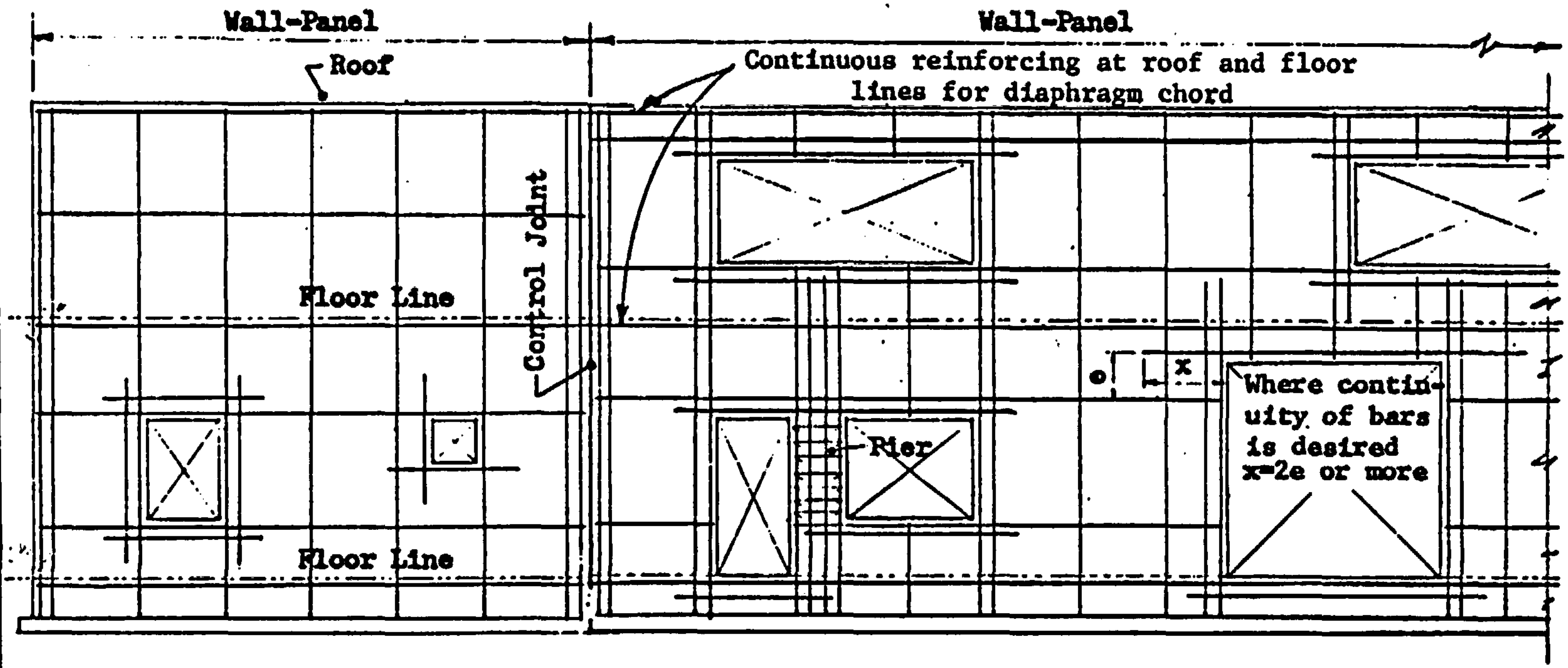
h. American National Standards Institute (ANSI), 1430 Broadway, New York, New York, 10018, ANSI-A41.1, "American Standard Building Code Requirement for Masonry."

i. National Concrete Masonry Association (NCMA), 2009-14th Street, N., Arlington, Virginia, 22202, "Specification for Design and Construction of Load-Bearing Masonry," (1971).

j. Structural Clay Products Institute (SCPI), 1750 Old Meadow Road, McLean, Virginia, 22101, "Building Code Requirements for Engineered Brick Masonry," (1969).

k. Plummer, Harry C., and Blume, John A., "Reinforced Brick Masonry and Lateral Force Design," Structural Clay Products Institute, 1750 Old Meadow Road, McLean, Virginia, 22101 (1953).

l. Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings," ATC 3-06, July 1978 (available from Government Printing Office, NBS SP-10).



ELEVATION OF A TYPICAL WALL

Figure 8-14.
Typical Wall Reinforcement
Reinforced Masonry

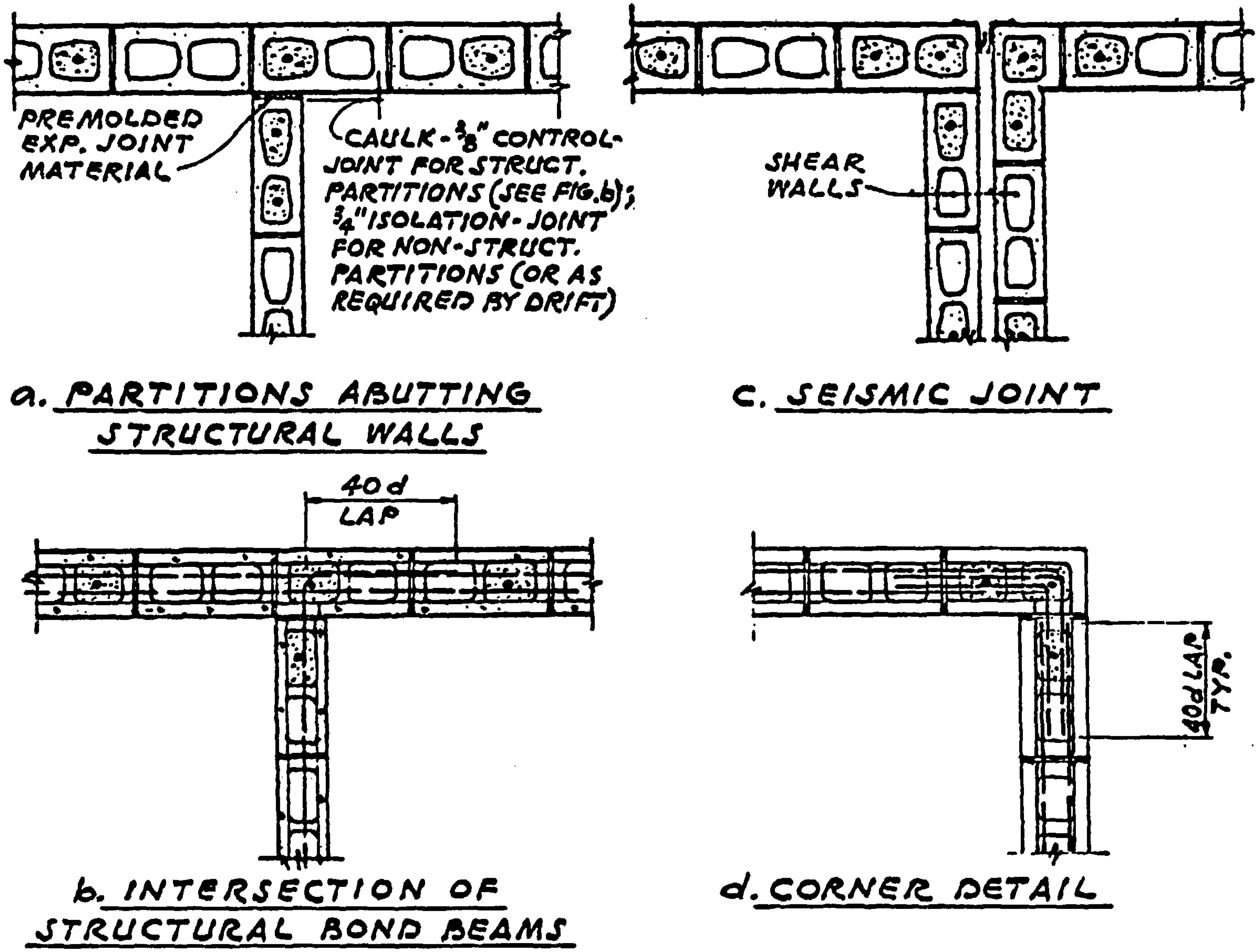


Figure 8-15. Intersection and Corner Details

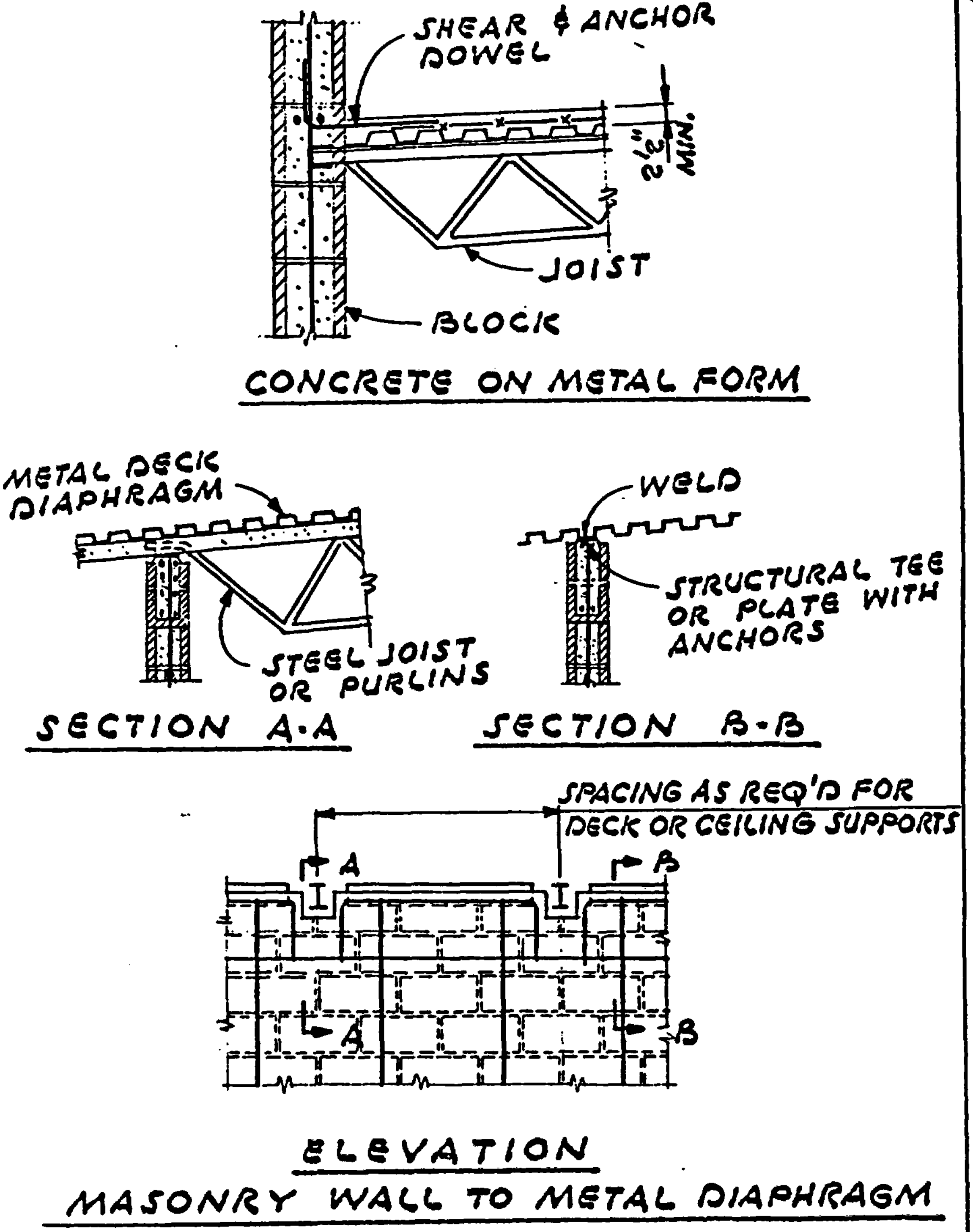


Figure 8-16. Miscellaneous Details

CENTERING AND CAGING DEVICES SHALL BE FORMED FROM #9 HARD STEEL WIRE, SPOT WELDED AND GALVANIZED. WATERSTOPS SHALL BE FORMED OF RUBBER OR POLYVINYLCHLORIDE.

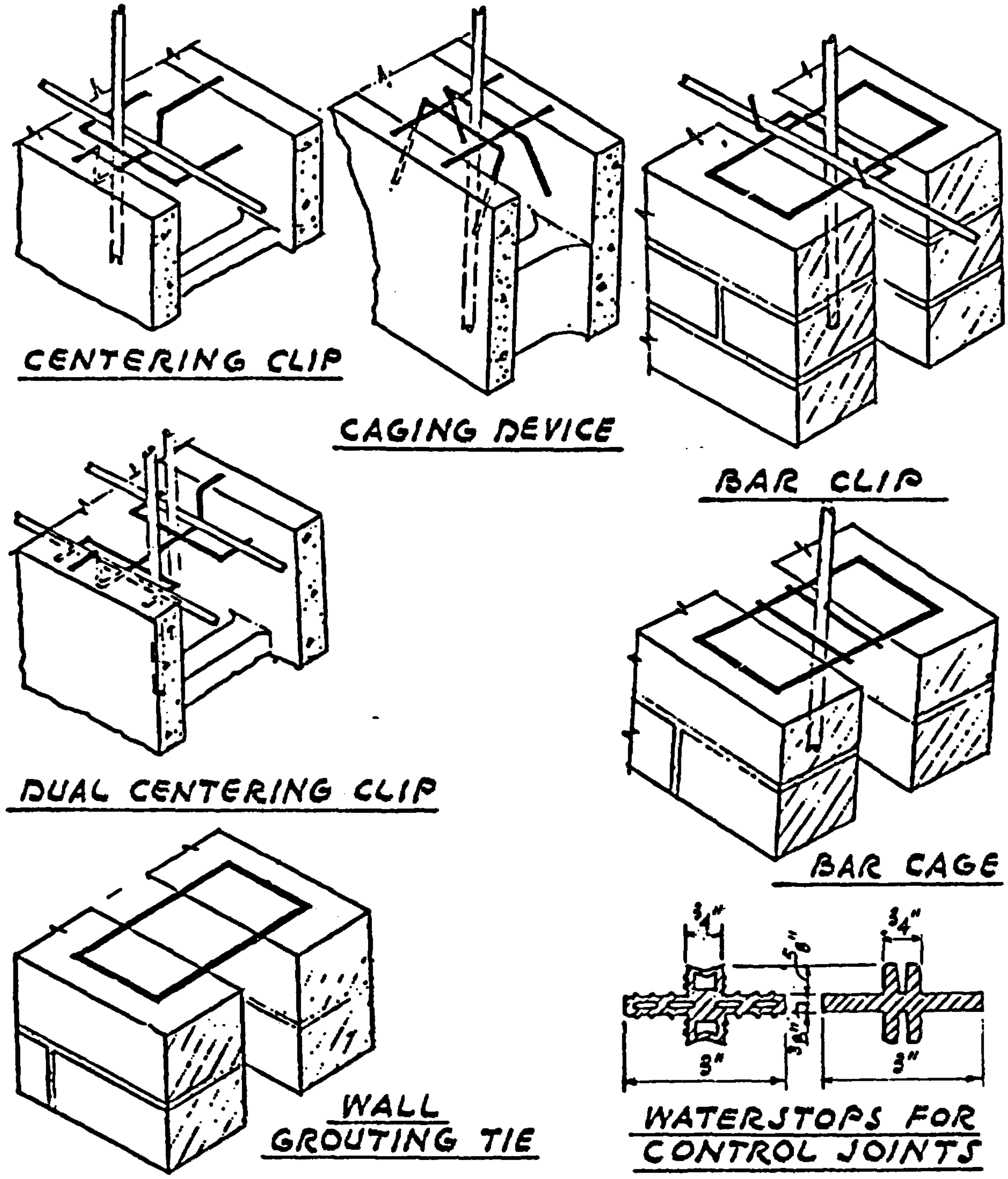


Figure 8-17. Accessories

Table 8-9. Average Weight of Concrete Masonry Unit
(2-Cell Unit, 8" x 8" x 16")

Thickness (inches)	Gross Area of Unit (square inches)	Net Area of Unit (square inches)	Lightweight Aggregate (pounds per unit)	Sand-Gravel Aggregate (pounds per unit)
4	57	37	15	20
6	88	50	23	33
8	119	57	28	38
12	182	83	40	56

Table 8-10. Average Weight of Completed Wall[1]
(Pounds per Square Foot of Wall)

Thickness (inches)		Lightweight Aggregate			Sand-Gravel Aggregate			Clay-block	Grouted Brick				
		6	8	12	6	8	12		9	9-1/2	10	11	12
Solid Grouted Wall		56	77	118	68	92	140	88	90	95	100	110	120
Spacing of Vertical Grouted Cores (inches)	16	46	60	90	58	75	111	71					
	24	42	53	79	53	68	99	64					
	32	40	50	73	51	65	93	61					
	40	38	47	70	50	62	89	58					
	48	37	46	68	49	61	87	55					

[1]A sand-gravel aggregate has been assumed for the grout and mortar. The above weights include an assumed average for bond beams and reinforcement.

CHAPTER 9
ARCHITECTURAL ELEMENTS

9-1. Purpose and scope. This chapter defines architectural elements, discusses their participation and importance in relation to the seismic design of the structural system, and prescribes the criteria for their design to resist damage from seismic lateral forces. The fundamental principle and underlying criterion of this chapter are that (a) the design of architectural elements will be such that they will not collapse and cause personal injury due to the accelerations and displacements induced by severe seismic disturbances, and (b) the architectural elements will withstand more frequent but less severe seismic disturbance without excessive damage and economic loss (refer to chap 2, para 2-9*k*). Mechanical and electrical elements are considered separately in chapter 10.

9-2. Definition. Architectural elements are generally defined as all elements of a building shown only in the architectural contract drawings (i.e., not detailed in the structural or mechanical/electrical drawings), such as nonstructural walls, partitions, windows, suspended ceilings, ornamentation, and appendages. A nonstructural architectural element is usually isolated or is so flexible such that it does not participate in the lateral shear resistance of the structure. For example, a wall which is isolated at the top and both ends, so as not to resist inplane deformations, is classified as a nonparticipating, nonstructural, architectural element. Note that such a wall must be braced laterally at the top or else it must cantilever from the floor (fig 9-1). A rigid nonbearing curtain or filler wall (e.g., concrete or masonry) that is not isolated, although generally considered as a nonstructural element, will obviously participate in shear resistance to horizontal forces parallel to the wall because it tends to deflect and be stressed when the building deforms under lateral forces. The degree of participation is dependent on the relative rigidities of such elements relative to the overall structure.

9-3. Design criteria. Architectural elements (1) must safely resist horizontal forces equal to a force coefficient times their own weight, and (2) must be capable of conforming (accommodating) to the lateral deflections that they will be subjected to during the lateral deformation of the building in which they are located.

a. Lateral Forces. The equivalent static lateral force that is applied to architectural elements is given by the formula 3-8 in chapter 3, paragraph 3-3(G),

$$F_p = Z\,I\,C_p\,W_p \qquad (3\text{-}8)$$

where the direction of the force F_p and the value of the coefficient C_p are prescribed in table 3-4. In general, the value of C_p is 0.30; however, for ornamentation, parapets, and other appendages, where the potential for collapse and injury is greater, C_p is 0.80. For exterior wall panels, C_p is 0.30; however, the special provisions of chapter 3, paragraph 3-3(J)3d apply.

b. Deflections. For the design of the structure, lateral deflections or drift of a story relative to its adjacent story is limited to 0.005 times the story height unless it can be demonstrated that greater drift can be tolerated (chap 3, para 3-3(H)1). The drift is calculated from the application of the required lateral forces multiplied by 1/K (1/K not less than 1.0).

(1) Architectural elements will be designed and detailed to conform to these drift requirements without damage.

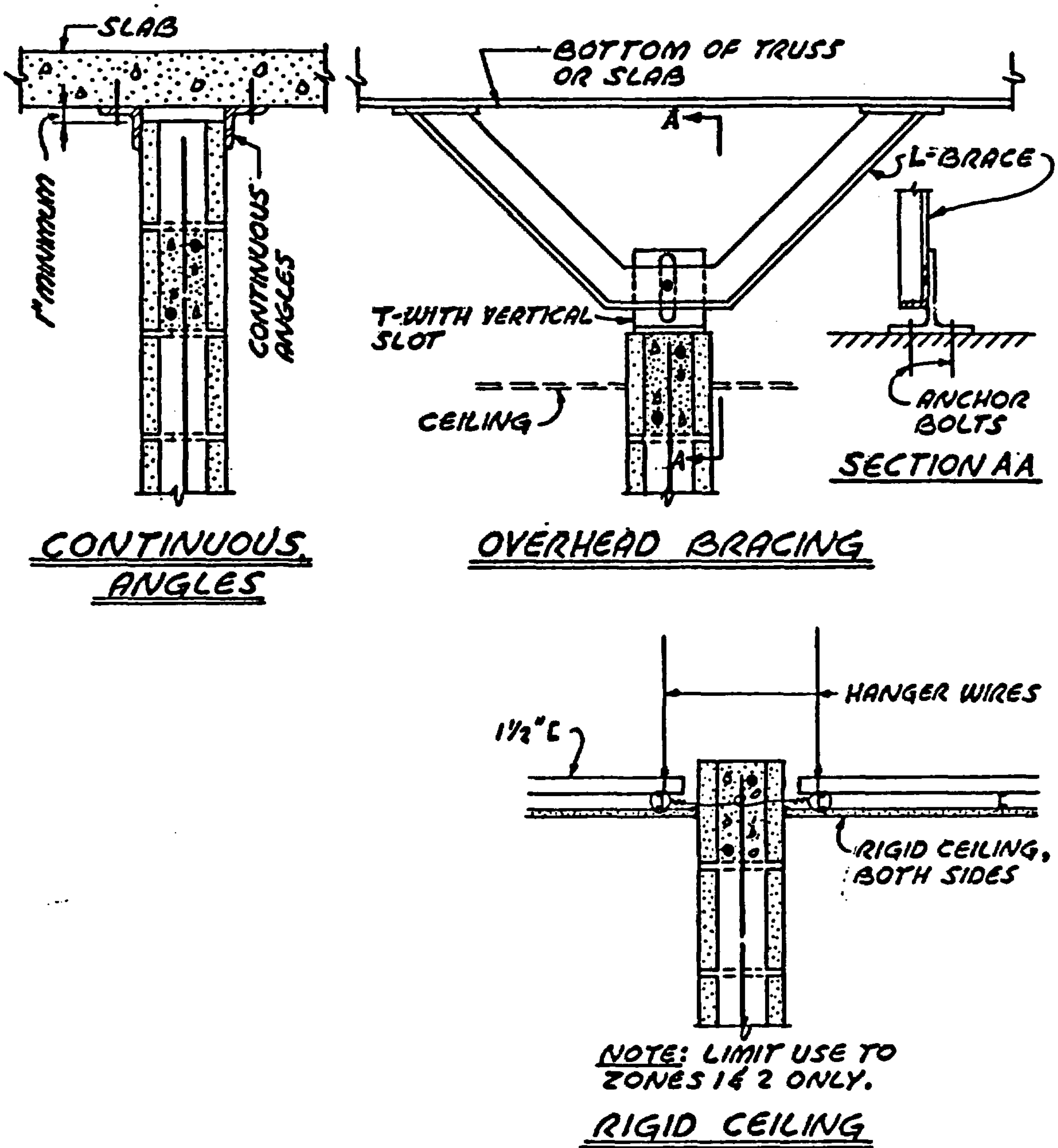
(2) Exterior elements are required to allow for relative movement equal to 3/K times the calculated elastic story displacement caused by required seismic forces or 1/2-inch per story, whichever is greater (chap 3, para 3-3(J)3d).

(3) The effects of adjoining rigid elements on the structural system will also be investigated (chap 3, para 3-3(J)1e).

9-4. Detailed requirements. *a. Partitions.* Partitions are classified into two general categories: (1) rigid and (2) nonrigid. Reference is also made to chapter 6, paragraph 6-2.

(1) *Rigid Partitions.* This category generally refers to nonstructural masonry walls. Where such a wall is unable to resist the lateral forces (parallel to its plane) that it is subjected to, based on relative rigidities, it will be isolated. Typical details for isolation of these walls are shown in figure 9-1. These walls will be designed for the prescribed forces normal to their plane.

(2) *Nonrigid Partitions.* This category generally refers to nonstructural partitions such as stud-and-drywall, stud and plaster, and movable partitions. When constructed according to standard recommended practice, it is assumed that the partitions can withstand the design inplane drift of 0.005 times the story height (i.e., 1/16 inch per foot of



Lateral Supports - Nonstructural Partition

Figure 9-1. Typical Details of Isolation of Walls height) without damage. Therefore, if the structure is designed to control drift within the prescribed limits, these partitions do not require special isolation details. They will be designed for the prescribed seismic force acting normal to flat surfaces. However, wind or the normal 10 pounds per square foot partition load will usually govern. If the structural design drift is not controlled within the prescribed limits, isolation of partitions will be required for reduction of nonstructural damage. Economic justification between potential damage and costs of isolation will be considered. Decision needs to be made for each project as to the role, if any, such partitions will contribute to damping and response of the structure, and the effect of seismic forces parallel to the partition resulting from the structural system as a whole. Usually, it may be assumed that this type of partition is subject to future alterations in layout location. The structural role of partitions may be controlled by height of partitions and methods of support.

b. Connections of Exterior Wall Panels. Precast, nonbearing, nonshear wall panels of other elements which are attached to, or enclose the exterior, will accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements will be supported by means of cast-in-place concrete or by mechanical devices. Connections and panel joints will be designed to allow for the relative movement between stories and will be designed for the forces specified in chapter 3, paragraph 3-3(J)3d. 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 otherwise terminated so as to effectively transfer forces to the reinforcing steel. Connections to permit movement in the plane of the panel for story drift may be properly designed sliding connections using slotted or oversize holes, or may be connections which permit movement by bending of steel components without failure. Typical design forces are shown in figure 9-2.

c. Suspended Ceiling Systems. Earthquake damage to suspended ceiling systems can be limited by proper support and detailing. Suspended ceiling framing systems in Seismic Zones 2, 3, and 4 will be designed for the prescribed forces in chapter 3, paragraph 3-3, table 3-4. The ceiling weight, W_p, shall include all light fixtures and other equipment which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used (reference table 3-4, footnote 6). The support of the ceiling systems will be by a positive means of support such as wire or an approved seismic clip system. Typical details of suspended acoustical tile ceilings are shown in figure 9-3.

d. Parapets, Ornamentation, and Appendages. These elements will be designed for forces resulting from C_p equal to 0.8 as prescribed in chapter 3, paragraph 3-3(G) and table 3-4. For the design of parapets refer to chapter 6, paragraph 6-2c.

e. Window Frames. Window frames will be designed to accommodate deflections of the structure without imposing a load on the glass. As glass is a brittle material, a considerable hazard of falling glass may be present. It is particularly serious if the glass is above and adjacent to a public way. This hazard can be eliminated by proper isolation between glass and its enclosing frame. It is obvious that the magnitude of isolation required depends upon the drift and the size of the individual pane or enclosing frame. Thus a pane of glass in a full story height frame should have an isolation or movement capability as great as the maximum possible drift (e.g., 3.0/K times the calculated elastic story displacement prescribed in chap 3, para 3-3(J)1d and 3-3(J)3d). The actual isolation clearance will depend on the geometry and deformation characteristics of enclosing frame, frame support, and structural system. Special care will be exercised in the field to see that such isolation is actually obtained.

f. Stairways. The rigidity of the stairway, relative to the structure, will be considered. In some cases the stairway will be isolated from the structure for lateral force considerations. Refer to chapter 4, paragraph 4-7*d*, for special seismic detailing.

g. Storage Racks. Chapter 3, paragraph 3-3(G), and table 3-4 prescribe the seismic design forces for storage racks. However, two alternative methods for determining the seismic design forces are permitted under certain conditions.

(1) *Table 3-4.* Lateral forces are determined from the formula $F_p = ZIC_pW_p$ (formula 3-8) where C_p is equal to 0.30 and W_p is equal to the weight of the racks plus contents. If the racks are self-supporting and located on the ground level of the building, C_p is reduced to a value of 0.20 (footnote 1 of table 3-4). If the racks are over two storage support levels in height, the C_p value for the storage levels below the top two levels is reduced by 20 percent (i.e., C_p equals 0.24, or 0.16 if self-supporting on the ground level).

(2) *Alternate No. 1.* Where a number of storage rack units are interconnected so that there are a minimum of four vertical elements in each direction on each column line designed to resist horizontal forces, the design coefficients may be as for a building with K values from table 3-3, CS = 0.20 for use in the formula V = ZIKCSW (formula 3-1) and W equal to the total dead load plus 50 percent of the rack rated capacity.

(3) *Alternate No. 2.* For pallet racks, drive-in and drive through racks, and stacker racks made of cold-formed or hot-rolled steel structural members which are located on the ground level of the building, the provisions of Uniform Building Code Standard N 27-11 may be used. This standard is based on "Interim Specifications for the Design, Testing, and Utilization of Industrial Steel Storage Racks," 1972, and "Supplement No. 1 to the Specification," June 18, 1973, by the Rack Manufacturers Institute (1326 Freeport Rd., Pittsburgh, PA 15238). These provisions are based on the formula V = ZIKCSW (formula 3-1), with the coefficients determined in a manner consistent with the provisions of chapter 3, paragraph 3-3, of this manual. W is equal to the weight of the racks plus contents.

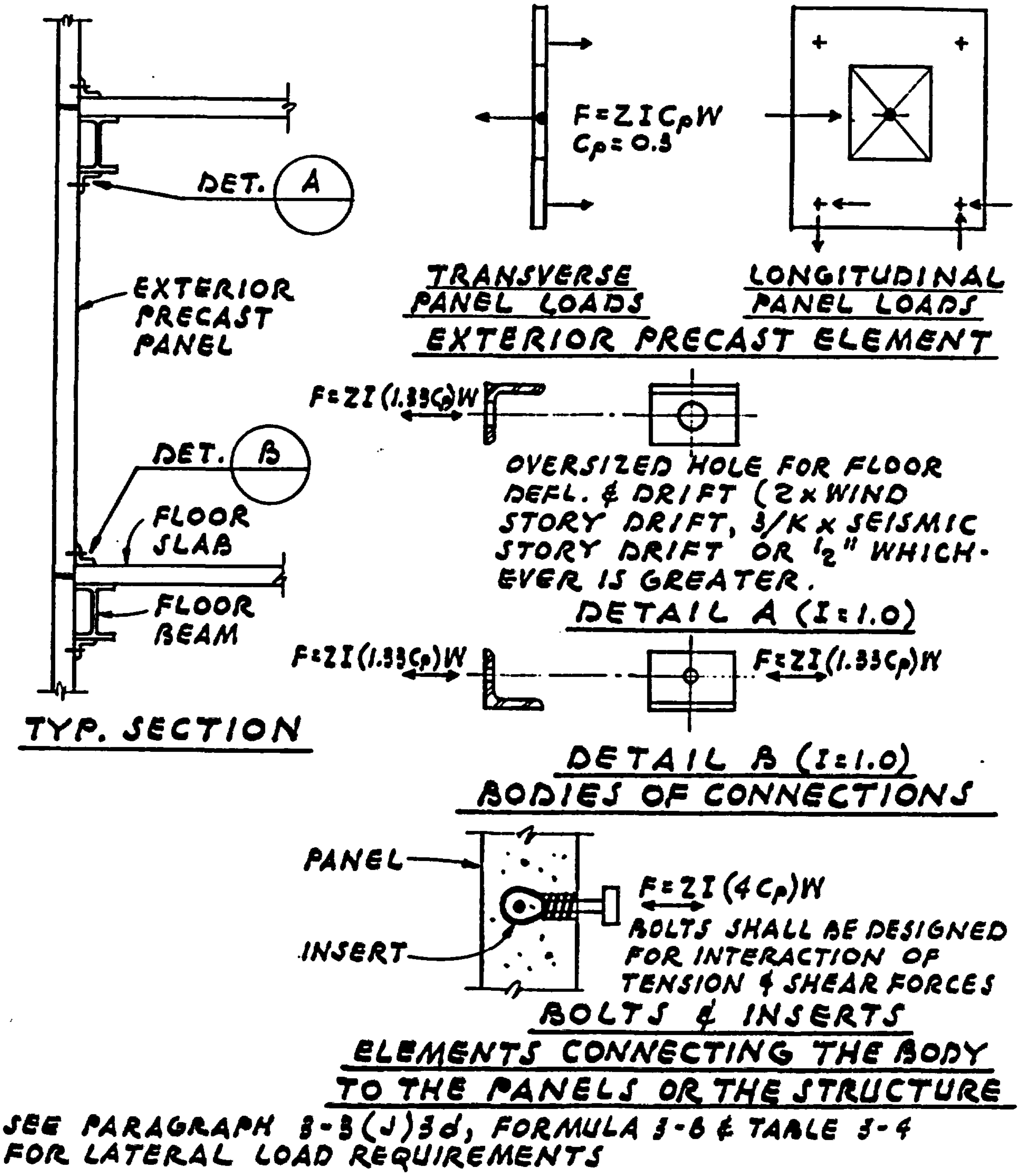
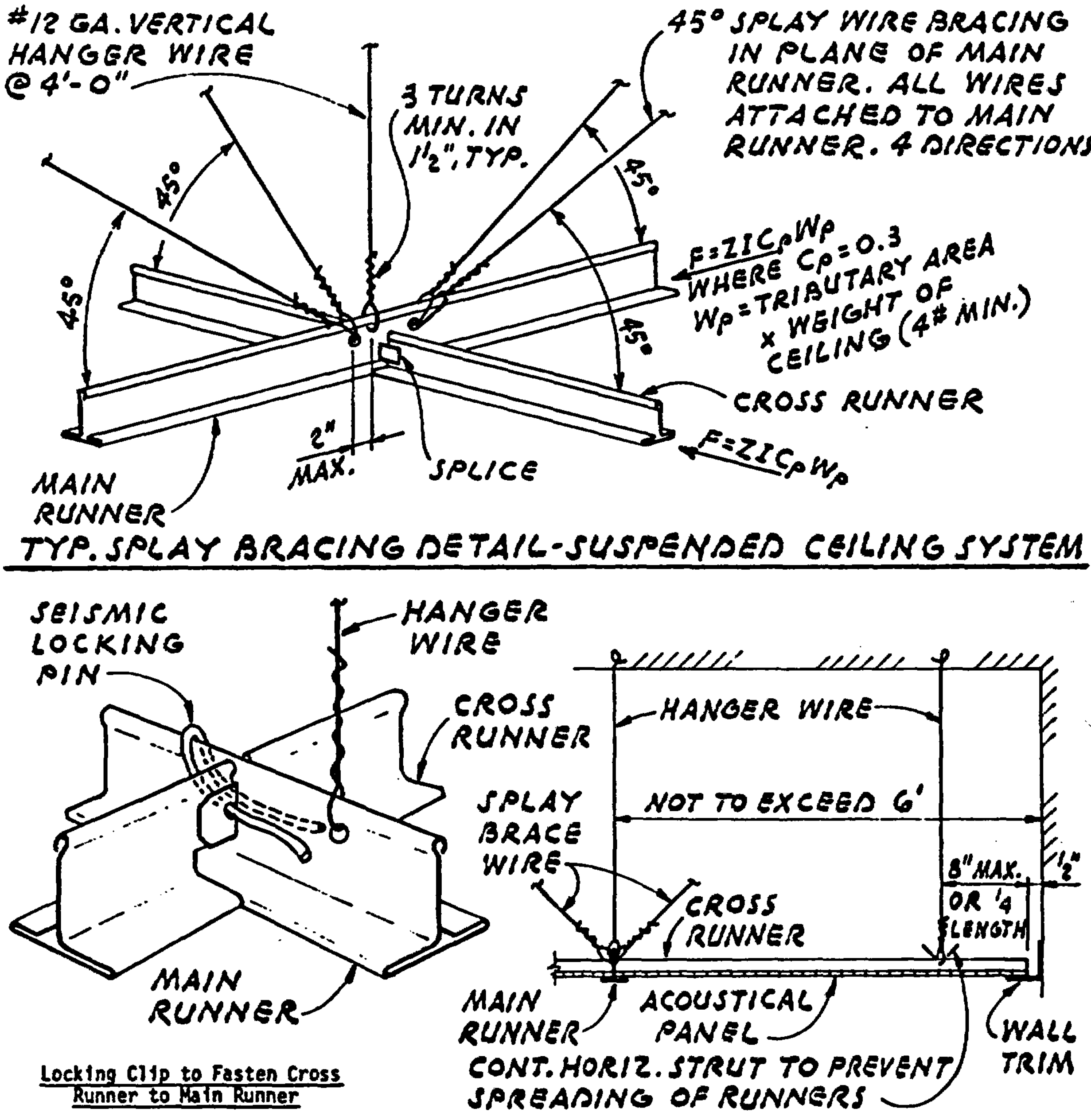


Figure 9-2. Design Forces for Exterior Precast Elements



Locking Clip to Fasten Cross Runner to Main Runner

Splices and intersections of runners shall be attached with mechanical interlocking connectors such as pop rivets, screws, pins, plates with bent tabs, or other approved connectors. Design connectors for 2 x design load or ultimate axial tension or compression (minimum 60 pounds).

HANGER WIRE	ALLOW. TENSION
#12	170#
11	225#
10	300#

Figure 9-3. Suspended Acoustical Tile Ceiling

CHAPTER 10
MECHANICAL AND ELECTRICAL ELEMENTS

10-1. Purpose and scope. This chapter prescribes the criteria for structural design of anchorages and supports for mechanical and electrical equipment in seismic areas. Mechanical and electrical equipment have been classified as being either ongrade or supported by the building and as being rigid or flexible. The principles and concepts given herein are intended to illustrate principles and concepts involved in seismic design of mechanical and electrical elements of buildings. The fundamental principle and underlying criterion of this chapter are that the design of mechanical and electrical element supports will be such that they will withstand (1) the accelerations induced by severe seismic disturbances without collapse or excessive deflection, and (2) the accelerations induced by less severe seismic disturbances without exceeding yield stresses. The design of the equipment itself is beyond the scope of this manual.

a. Modification to SEAOC Approach. The seismic force criteria for rigid and rigidly mounted equipment are generally covered by the SEAOC provisions in chapter 3, paragraph 3-3(G), and table 3-4. In order to fulfill the requirements of mechanical and electrical elements not specifically covered by chapter 3, a modification to the SEAOC approach is presented in this chapter. Particular attention is given to criteria for the estimation of horizontal force factors on flexible and flexibly mounted equipment.

b. Seismic Forces. The design forces applied to equipment supports are generally higher than the forces used in the design of buildings. One reason is the amplification of the ground motion acceleration transmitted to elements in the elevated stories of a building due to dynamic response. Another reason is equipment supports often lack the extra margin of safety provided by reserve strength mechanisms, such as participation of architectural elements, inelastic behavior of structural elements, and redundancy in the structural system, that are characteristic of buildings.

10-2. General requirements. All equipment supports designed under the provisions of this chapter, for either equipment on the ground or in buildings, will conform to the following requirements:

a. Rigid Equipment and Rigid Supports. Rigid equipment that is rigidly attached to the structure or to the ground will be designed for seismic forces prescribed by chapter 3, paragraph 3-3(G), of this manual. Limitations, exceptions, and commentary are stated in paragraphs 10-3 and 10-5 below.

b. Flexible Equipment or Equipment on Flexible Supports. For flexible and flexibly mounted equipment and machinery, chapter 3, paragraph 3-3(G) and table 3-4, footnote 3, state that the appropriate values of 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. As an alternative to a rigorous analysis, a procedure is outlined in paragraph 10-4 to obtain horizontal design seismic forces for flexible or flexibly mounted equipment (and machinery) located in the building. Paragraph 10-5 discusses the criterion for locations on the ground.

c. Weight Limitations. Equipment in buildings will be considered to be within the scope of this chapter if:

(1) The maximum weight of the individual item of equipment does not exceed 10 percent of the total building weight.

(2) The maximum weight of the individual item of equipment does not exceed 20 percent of the total weight of the floor at the equipment level.

The response of equipment is dependent upon the response of the building in which it is housed. If the weight of the equipment is appreciable, relative to the weight of the building, the interaction of the equipment with the building (i.e., coupling effect) will change the building response characteristics. It is assumed that equipment within the above weight limitations has a negligible effect on the response of the building. Equipment that is not within the above limitations is outside the scope of this manual and must be designed using a more rigorous method of analysis.

d. Rigorous Analysis. No portion of this chapter will be construed to prohibit a rigorous analysis of equipment and the supporting mechanism by established principles of structural dynamics. Such an analysis will demonstrate that the fundamental principle and underlying criterion of paragraph 10-1 are satisfied. In no case will the design result in capacities less than 80 percent of those required by chapter 3, paragraph 3-3(G).

e. Combined States of Stress. Combined states of stress, such as tension and shear on anchor bolts, will be investigated in accordance with established principles of structural engineering. Refer to chapter 4, paragraph 4-6*d*.

f. Securing Equipment. Use of friction as a method of resisting seismic forces is not acceptable and will not be allowed. Both vertical and horizontal accelerations are possible during an earthquake. Under vertical acceleration, the normal force required to maintain friction can be greatly diminished. This could result in a reduction of the friction force available to resist horizontal seismic loads as both simultaneous vertical and horizontal accelerations are possible. Thus, equipment will be secured by bolts, embedment, or other acceptable positive means of resisting horizontal forces. Refer to paragraph 10-11 for example of typical details.

g. Special Requirements. Additional requirements for lighting fixtures and supports, piping, stacks, bridge cranes and monorails, and elevator systems are covered in paragraphs 10-6 through 10-10, respectively.

10-3. Rigid and rigidly mounted equipment in buildings. Rigid and rigidly mounted equipment will be considered to be those equipment units and equipment supporting systems for which the period of vibration as defined in paragraph 10-4b is estimated to be less than 0.05 second (i.e., frequency of vibration greater than 20 Hz). Compact equipment directly attached to a concrete pad or a footing will be considered rigidly supported. This type of equipment-supporting system is very stiff, and the period of vibration is very short (i.e., high frequency of vibration). Equipment not satisfying the rigidity requirement will be designed according to the criteria of paragraph 10-4.

a. Examples of Rigidly Mounted Equipment.

(1) A boiler bolted or otherwise securely attached to a concrete pad or directly to the floor of a structure.

(2) An electrical panel board securely attached to solid walls or to the studs of stud walls.

(3) An electric motor bolted to a concrete floor.

(4) A floodlight having a short stem bolted to a wall.

(5) A rigidly anchored heat exchanger.

b. Equivalent Static Force. The equivalent static lateral force is given by formula (3-8) in chapter 3, paragraph 3-3(G).

$$F_p = Z I C_p W_p \qquad (3\text{-}8)$$

C_p, as prescribed in table 3-4, is equal to 0.30 for all equipment and machinery that are rigid and rigidly attached to the building (see para 10-5 for equipment on the ground). For cantilevered portion chimneys and smokestacks, C_p is 0.80; however, these items must also be investigated for the criterion stated in paragraph 10-8.

10-4. Flexible equipment or flexibly mounted equipment in buildings. Equipment that does not satisfy the rigidity requirements of paragraph 10-3 will be considered to be flexible or flexibly mounted. For flexible and flexibly mounted equipment (and machinery), the appropriate seismic design forces will 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 (chap 3, table 3-4, footnote 3). An approximate procedure, which considers these dynamic properties within certain limits, is presented below. Flexible or flexibly mounted equipment that does not qualify within the limits of this chapter is outside the scope of this manual and will be designed using a more rigorous method of analysis.

a. Single Mass System. The approximate procedure is based on the equipment responding as a single-degree-of-freedom system to the motion of one of the predominant modes of vibration of the building at the floor level in which the equipment is placed. Therefore, if the equipment and its supporting system cannot be approximated by a single-degree-of-freedom system (i.e., a simple oscillator represented by a single mass and a simple spring), a more rigorous analysis is required. Some examples of systems that do qualify under this procedure follow:

(1) Rigid equipment attached to the floor slab with a spring isolation system.

(2) Rigid equipment, rigidly attached to a flexible supporting system that is rigidly attached to the floor slab.

(3) Rigid equipment attached by a cantilever support from the structure.

(4) Flexible equipment, which can be represented as a single mass system, rigidly attached to the structure.

EXCEPTIONS: Equipment that can be considered to have uniformly distributed mass will be designed for seismic forces in a manner similar to stacks (para 10-8). Lighting fixtures, piping, stacks, bridge cranes and monorails, and elevator systems will be designed as specified in paragraphs 10-6 through 10-10, respectively.

b. Equipment Period Estimation. For equipment responding as a single-degree-of-freedom sy[stem] the period of vibration, T_a, is equal to $2\sqrt{ma}$...

stiffness. In terms of inch and pound units, this formula becomes

$$T_a = 2\pi\sqrt{\frac{W/g}{k}} = \frac{2\pi}{\sqrt{386}}\sqrt{\frac{W}{k}} = 0.32\sqrt{\frac{W}{k}} \qquad (10\text{-}1)$$

where

T_a = Fundamental period (sec).
k = Stiffness of supporting mechanism in terms of load per unit deflection of the center of gravity (lb/in.).
W = Weight of equipment and/or equipment supports (lb), which is equal to the mass times the acceleration of gravity.
g = Acceleration of gravity at 386 in./sec^2.

In lieu of calculating the period of vibration using Equation 10-1, a properly substantiated experimental determination will be allowed.

c. *Building Period Estimation.* If a building has more than one story it is considered to be a multi-degree-of-freedom system with more than one mode of vibration. Flexible equipment located in the building can be excited to respond to any of the predominant modes of the building vibration. Therefore, when investigating the response of equipment to the floor motion response, all predominant modes of vibration must be considered. The building periods will be based on realistic estimations that are not restricted to limitations used in building design criteria.

(1) *Fundamental mode of vibration.* The fundamental period of the building vibration T_1 corresponds to the period T used in the design of the building. A realistic estimation of T_1 will probably lie somewhere between the value used to determine the force coefficients (chap 4, para 4-3*d*) and the value used to determine the drift compliance (chap 4, para 4-5c).

(2) *Higher modes of vibration.* In addition to the fundamental mode of vibration, the predominant higher modes of vibration must be considered.

(a) For regular structures (section 3-3(E)), with fundamental periods less than 2 seconds, include the second and third modes of vibration (translational modes in the direction under consideration). In lieu of a detailed analysis, the second mode period of vibration may be assumed to equal 0.30 times the fundamental period of vibration (i.e., $T_2 = 0.30\ T_1$) and the third mode period of vibration may be assumed to equal 0.18 times the fundamental period of vibration (i.e., $T_3 = 0.18\ T_1$).

(b) For buildings with fundamental periods greater than 2 seconds, the fourth mode and possibly the fifth mode should also be included.

(c) For irregular buildings the dynamic characteristics of the structure must be investigated to determine other (nontranslational or torsional) predominant modes.

(d) In some cases, the vertical modes of vibration should be considered. This applies to floor systems that are flexible in the vertical direction and equipment sensitive to vertical accelerations.

d. *Appendage Magnification Factor.* The appendage magnification factor (M.F.) is the ratio of the peak motion of the appendage (in this case, equipment) to the peak motion of the floor level that it is mounted on. A theoretical value of the M.F. is generally based on steady-state motion due to the floor responding as a uniform sine wave. However, buildings that are responding to earthquakes move in a somewhat random fashion and thereby do not generate magnification factors as large as calculated by theoretical steady-state responses. Following are discussions on the steady-state response and on an approximate method for estimating appendage magnification factors.

(1) The magnification factor for an idealized single mass oscillator, with a period T_a and damping characteristics at 2 percent of critical damping, responding to a steady-state sinusoidal acceleration having a period T, is plotted on figure 10-1. If T_a is essentially equal to T, M.F. equals 25. In other words, at a condition of resonance, the maximum acceleration of the oscillator mass will be 25 times the peak acceleration of the forcing motion. This idealized condition depends on (a) fine tuning of the two periods, (b) linearity of the oscillator spring, (c) uniformity of the input sinusoidal motion, and (d) length of time of the input motion (at least 25 cycles).

(2) If the oscillator represents the equipment, the floor response represents the steady-state input motion, and the C_p value of 0.30 is assumed to be the floor acceleration, the peak acceleration for the equipment is 25 times 0.30g = 7.5g. In other words, the horizontal force on the equipment is seven and one-half times its own weight. However, due to the actual nonlinear characteristics of equipment and buildings and particularly the finite duration of earthquake motion, it is highly unlikely that such a magnification could actually occur to a 2 percent damped equipment appendage.

(3) In order to approximate a realistic value for a design M.F. factor, it is assumed that (a) the periods T_a and T will differ by at least 5 percent; (b) buildings are not perfectly linear elastic, especially at high amplitudes of response; (c) the floor response is not an exact, uniform sine wave; and (d) the number of high amplitude floor response cycles is substantially less than 25.

(4) The design M.F. factor curve shown in figure 10-2 is presented as an aid to estimating the de-

sign response of single-degree-of-freedom appendages, in lieu of more rigorous analysis methods. The peak M.F. of 25 is reduced to 7.5 by reducing the effectiveness of the period tuning, the peak floor response amplitude, and the number of continuous cycles to roughly two-thirds of the idealized values (i.e., 25 × 2/3 × 2/3 × 2/3 ≅ 7.5). The width of the magnification factor is broadened to account for uncertainty of actual period ratios.

e. Equivalent Static Force. The equivalent static force for the anchorage of flexible and flexibly mounted equipment is given by the formula

$$F_p = Z I A_p C_p W_p \quad (10\text{-}2)$$

which is a modification of the rigid equipment formula 3-8, where A_p is the amplification factor for the coefficient C_p. The value of A_p is related to the M.F. values of figure 10-2; however, the maximum value of 7.5 is reduced to a value of 5.0 to account for multimode effects that are assumed to be included in the C_p values of table 3-4 (i.e., the C_p for rigid equipment considers the peak floor acceleration for a combination of modes; however, only one of these modes will excite the single resonance frequency of the flexibly mounted equipment). The value of A_p will be determined by one of the alternatives listed below:

(1) If the periods of the building and equipment are not known, $A_p = 5.0$.

(2) If the fundamental period of the building is known (see para 10-4*c*(1)), but the period of the equipment is not known, A_p is determined by table 10-1.

(3) If building and equipment periods are both known, A_p may be approximated by the graph in figure 10-3.

f. Use of the Equivalent Static Force Procedure. The force F_p of formula 10-2 will be applied in the same manner as the force F_p for rigid equipment in chapter 3, paragraph 3-3(G). As an aid to determining the A_p value, the following examples are given.

(1) *A standard anchorage system is to be signed for some flexible equipment that will placed in several buildings.* In order to have one universal anchorage system that will apply to all buildings, use A_p equal to 5.0.

(2) *An anchorage system is to be designed for some flexible equipment that will be placed in a building with a fundamental period of less than 0.5 seconds.* Because the period of the equipment is not given, use table 10-1. $A_p = 5.0$.

(3) *An anchorage system is to be designed for some flexible equipment that will be placed in a building with a fundamental period of roughly 1.4 seconds.* Because the period of the equipment is not given, use table 10-1. Interpolate between 1.0 second and 2.0 seconds. $A_p = 3.7$.

(4) *An anchorage system is to be designed for equipment with a period T_a equal to 0.2 second.*

(a) *In a building with T = 0.5 second.* Because both the building period and equipment period are known, use figure 10-3(a). $T_a/T = 0.2/0.5 = 0.4$ and $A_p = 2.7$.

(b) *In a building with T = 1.4 seconds.* Use figure 10-3(b). $T_a/T = 0.2/1.4 = 0.14 < 1.2$. Thus, A_p is equal to the value in Table 10-1; $A_p = 3.7$.

(5) *An anchorage system is to be designed for equipment with a period T_a equal to 2.0 seconds.*

(a) *In a building with T = 0.5 second.* figure 10-3(a). $T_a/T = 2.0/0.5 = 4.0$; $A_p = 1.0$.

(b) *In a building with T = 1.4 seconds.* Use figure 10-3(b). $T_a/T = 2.0/1.4 = 1.4$. Interpolate between the curves for T = 1.0 seconds and T = 2.0 seconds. $A_p = 3.0$.

g. Lateral Bracing. Stiffening of the equipment supports by lateral bracing may be used to reduce the appendage period; thus, possibly reducing the design seismic loads. Lateral bracing for compression members expressly designed for seismic forces will not exceed the slenderness limitation of L/r < 200 in any direction. L is the unbraced length in

*Table 10-1. Amplification Factor, A_p, for Flexible or Flexibly Mounted Equipment**

Building period t, sec	Less than 0.5	0.75	1.0	2.0	Greater than 3.0
A_p	5.0	4.75	4.0	3.3	2.7

*The values for A_p are based on a modal analysis using the period estimates of paragraph 10-4c, the design magnification factors of paragraph 10-4d, and a fairly standard response spectrum shape. The values in table 10-1 apply only to regular structures or framing systems (chap 3, para 3-3(E)).

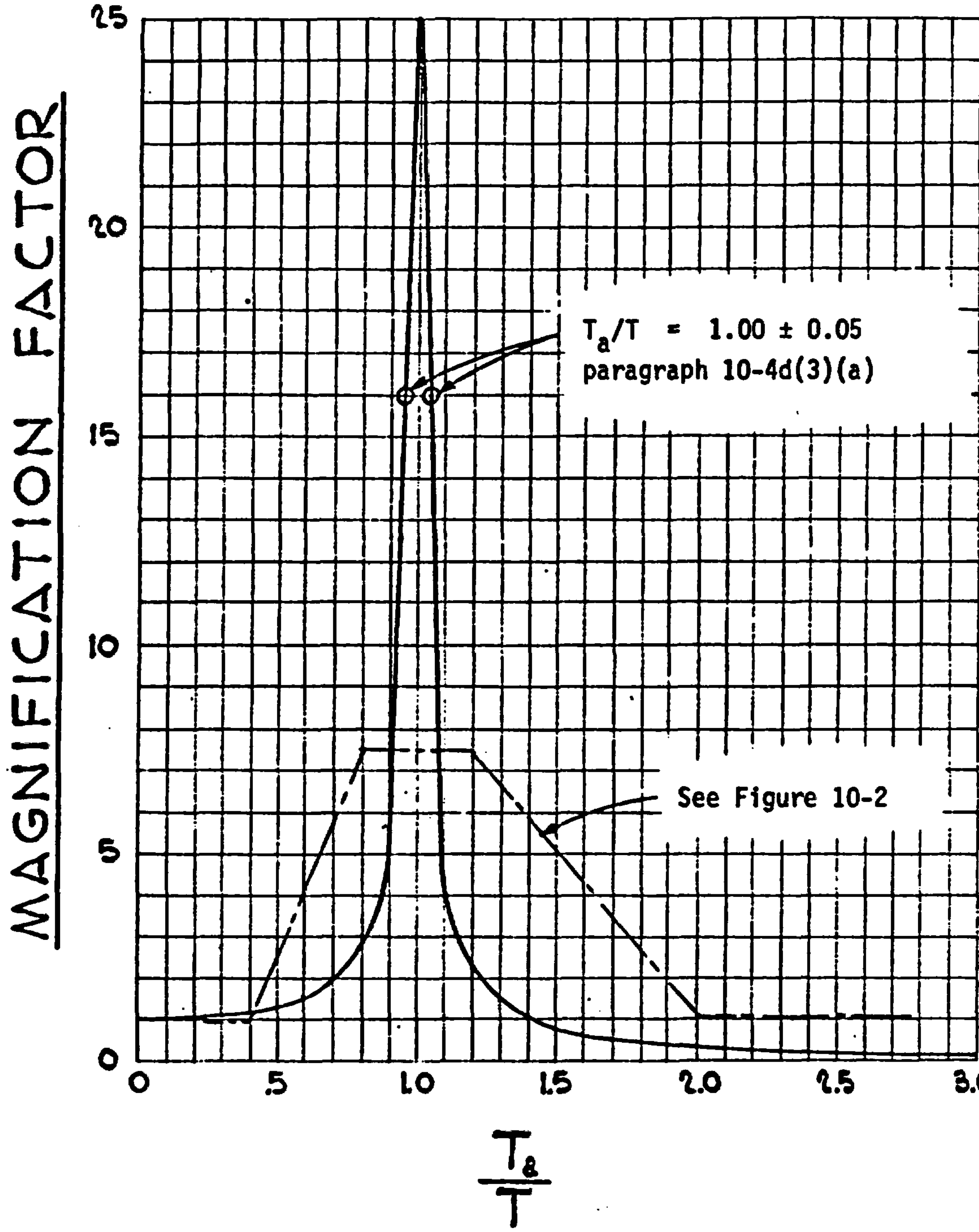


Figure 10-1. Idealized Acceleration Magnification Factor vs Period Ratio at 2% of Critical Damping

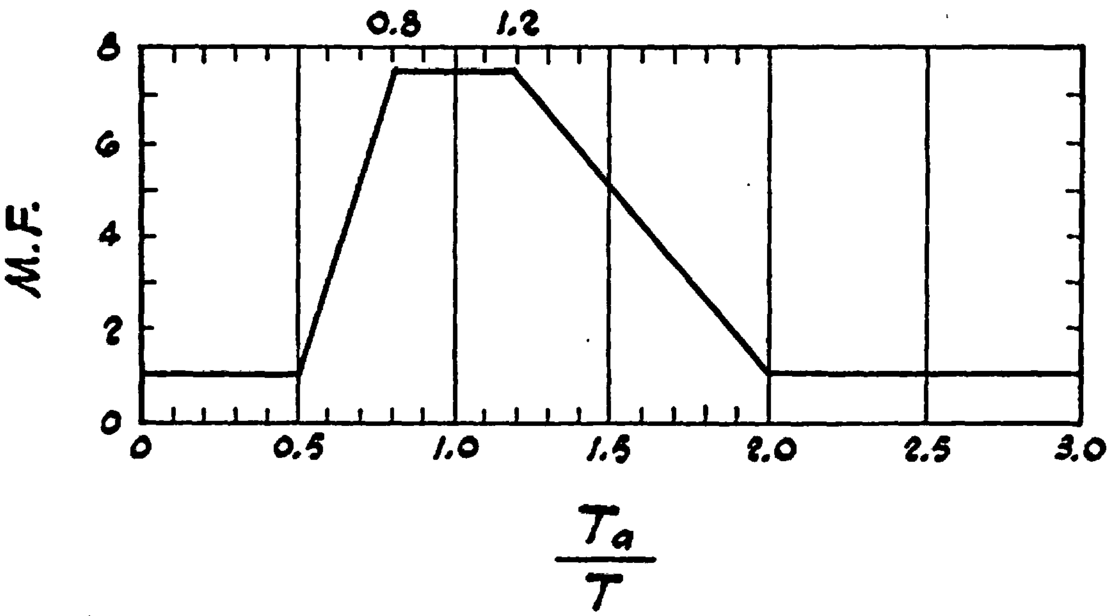
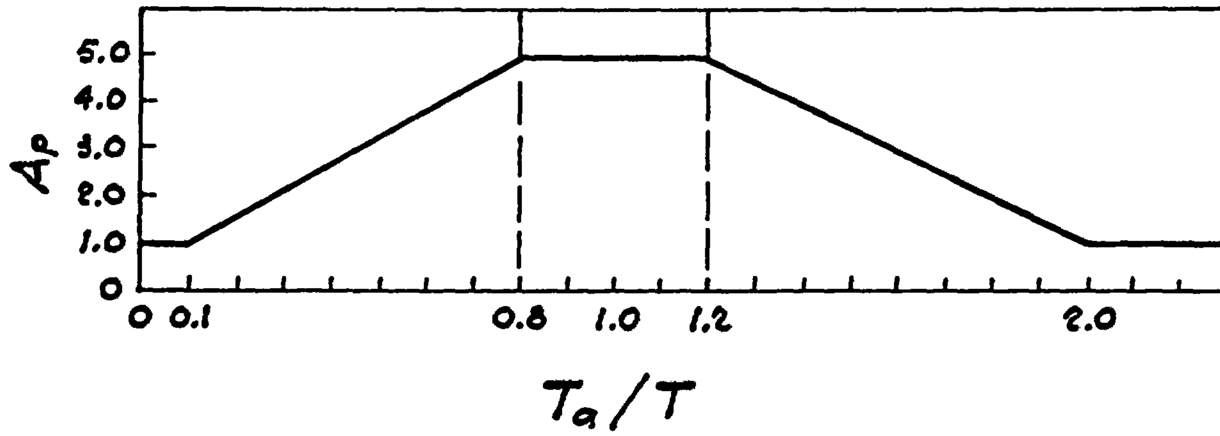
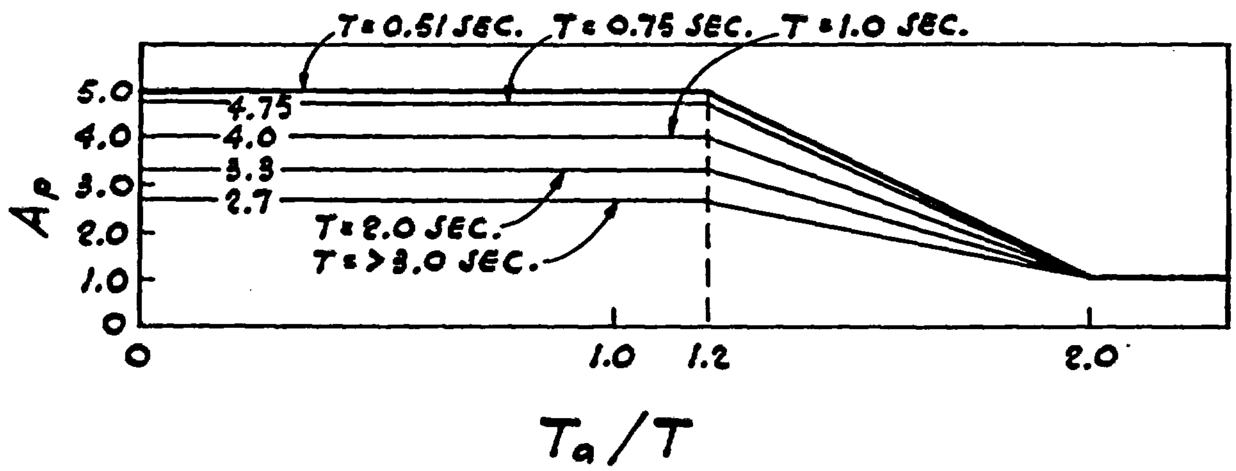


Figure 10-2. Design M.F. vs Period Ratio



(a) When the fundamental period of the building is equal or less than 0.5 seconds ($T \leq 0.5$).



(b) When the fundamental period of the building is greater than 0.5 seconds ($T > 0.5$). (Note: If $T_a/T < 1.2$, A_p is equal to value obtained from Table 10-1.)

Figure 10-3. Amplification Factor, A_p, for Flexible and Flexibly Mounted Equipment (Footnote to Table 10-1 applies) inches in the direction considered and r is the corresponding radius of gyration in inches.

h. Storage Tank Hydrodynamic Effects. Storage tanks in which the liquid is rigidly contained need not have hydrodynamic effects included in the seismic design when using the equivalent static force procedure. However, when the sloshing effects of the liquid could be detrimental to the function of the tank, the hydrodynamic effects will be considered. Refer to chapter 11, paragraph 11-4, for guidance in utilizing established principles of fluid mechanics and structural dynamics.

10-5. Equipment on the ground. Equipment classified as equipment on the ground will be that equipment in contact with or buried in the soil; that equipment supported by means of a slab, footing, or pedestal directly supported by the soil or on piles embedded in the soil; or equipment which is mounted on a tower, pole, or other similar structure that is soil-supported. Such equipment may be classified in one of three general categories, depending on its size, shape, and dynamic characteristics. The general categories are: (a) rigid and rigidly mounted equipment; (b) flexible or flexibly mounted equipment; and (c) large complex equipment or equipment on large or complex supports that are classified as structures other than buildings (chapter 11).

a. Rigid and Rigidly Mounted. Rigid and rigidly mounted equipment located on the ground are defined in the same manner as equipment considered in paragraph 10-3 except that the weight limitation need not be considered. The equivalent static lateral force is given by the formula

$$F_p = Z\,I\,(2/3\,C_p)\,W_p \qquad (10\text{-}3)$$

as prescribed by chapter 3, paragraph 3-3(G). C_p is prescribed in table 3-4. The two-thirds reduction factor applies for equipment and machinery supported at ground level that is rigid and is rigidly attached (table 3-4, footnote 1).

b. Flexible or Flexibly Mounted. Flexible or flexibly mounted equipment located on the ground responds to seismic motion in a similar manner that a structure responds to seismic motion. Such equipment is generally not subjected to the additional magnification factors of similar equipment located in the elevated stories of buildings. Equipment considered in this paragraph is limited to that which can be approximated by a single degree-of-freedom system (para 10-4*a*). The equivalent static lateral force is given by the formula

$$F_p = Z\,I\,(2\,CS)\,W_p \qquad (10\text{-}4)$$

or by Formula 10-3 in paragraph a, abo whichever is larger. C and S will be determined prescribed in chapter 3, paragraph 3-3, except that the equipment period T_a (para 10-4*b*) will be used in lieu of the building period T. When the periods are unknown, (2 CS) will be equal to the maximum value of 0.28.

c. Equipment Classified as Structures Other Than Buildings. For large or complex equipment, or when equipment is supported by a large or complex structure, the equipment and support system are classified as structures other than buildings and their seismic design is governed by the provisions in chapter 11, Structures Other Than Buildings. Example of equipment that are classified in this category are large pole mounted transformers (Design Example F-2), a missile tracking device situated on a truss tower (Design Example F-3) and large stacks or chimneys supported on the ground. The equivalent static lateral force criteria is given by formula 3-1 in chapter 3, paragraph 3-3(D).

$$F_p = V = Z\,I\,K\,C\,S\,W \qquad (3\text{-}1)$$

where K is equal to 2.0 or 2.5 as prescribed in table 3-3 and in chapter 11. Distribution of lateral forces will be in accordance with chapter 3, paragraph 3-3(E). For systems with uniform mass distribution, such as stacks and chimneys, refer paragraph 10-8 for distribution of lateral forces.

10-6. Lighting fixtures in buildings. In addition to the requirements of the preceding paragraphs, lighting fixtures and supports will conform to the Standards for Safety UL-57 and requirements given hereinafter.

a. Materials and Construction.

(1) Fixture supports will employ materials which are suitable for the purpose. Cast metal parts, other than those of malleable iron, and cast or rolled threads will be subject to special investigation to assure structural adequacy.

(2) Loop and hook or swivel hanger assemblies for pendent fixtures shall be fitted with a restraining device to hold the stem in the support position during earthquake motions. Pendent supported fluorescent fixtures shall also be provided with a flexible hanger device at the attachment to the fixture channel to preclude breaking of the support. The motion of swivels or hinged joints shall not cause sharp bends in conductors or damage to insulation.

(3) Each recessed fluorescent individual continuous row of fixtures shall be supported b seismic resistant suspended ceiling support system

and shall be fastened thereto at each corner of the fixture; or shall be provided with fixture support wires attached to the building structural members using two wires for individual fixtures and one wire per unit of continuous row fixtures. These support wires (minimum No. 12 ga. wire) will be capable of supporting four times the support load.

(4) A supporting assembly which is intended to be mounted on an outlet box will be designed to accommodate mounting features on four-inch boxes, three-inch plaster rings, and fixture studs.

(5) Each surface mounted fluorescent individual or continuous row of fixtures shall be attached to a seismic resistant ceiling support system. Fixture support devices for attaching to suspended ceilings shall be a locking type scissor clamp or a full loop band which will securely attach to the ceiling support. Fixtures attached to underside of a structural slab shall be properly anchored to the slab at each corner of the fixture.

(6) Each wall mounted emergency light unit shall be secured in a manner to hold the unit in place during a seismic disturbance.

b. Tests. In lieu of the requirements for equipment supports given in paragraph 10-4, lighting fixtures and the complete fixture supporting assembly may be accepted by passing shaking table tests approved by the using agency. Such tests will be conducted by an approved and independent testing laboratory, and the results of such test will specifically state whether or not the lighting fixture supports satisfy the requirements of the approved tests. Suspension systems for light fixtures, as installed, that are free to swing a minimum of 45° from the vertical in all directions and will withstand, without failure, a force of not less than four times the weight it is intended to support will be acceptable.

10-7. Piping in buildings. Pipes are categorized as either (a) pipes related to fire protection, (b) pipes not requiring seismic restraints, or (c) service pipes not related to fire protection.

a. Fire Protection Systems. All water pipes for fire protection systems will be designed under the provisions of the current issue of the "Standard for the Installation of Sprinkler Systems" of the National Fire Protection Association (NFPA No. 13).

(1) *Justification.* Pipes designed under NFPA No. 13 have performed satisfactorily during earthquakes. To avoid possible conflict in some areas with the NFPA recommendations, the criteria established in the following paragraphs will not be made applicable to piping expressly designed for fire protection. Designers of fire protection systems will thus obtain a more unified approach to seismic design; one which will be consistent with all NFPA requirements.

b. Pipes and Ducts That Do Not Require Special Seismic Restraints. Seismic restraints may be omitted from the following installations: (Exception: For essential facilities, critical piping will be designed in accordance with para c.)

(1) Gas piping less than 1-inch inside diameter.

(2) Piping in boiler and mechanical equipment rooms less than 1-1/4 inches inside diameter.

(3) All other piping less than 2-1/2 inches inside diameter.

(4) All electrical conduit less than 2-1/2 inches inside diameter.

(5) All rectangular air handling ducts less than 6 square feet in cross sectional area.

(6) All round air handling ducts less than 28 inches in diameter.

(7) All piping suspended by individual hangers 12 inches or less in length from the top of pipe to the bottom of the support for the hanger.

(8) All ducts suspended by hangers 12 inches or less in length from the top of the duct to the bottom of the support for the hanger.

c. Pipes Not Related to Fire Protection. Piping not governed by paragraph a. or b. above will be designed in accordance with the applicable following provisions.

(1) *General.* The provisions of this paragraph apply to the following:

(a) Risers. All risers and riser connections. See paragraph 10-7c(2) for design provisions and design example figure 9, Water Risers.

(b) Horizontal pipe. All horizontal pipes and attached valves. For the seismic analysis of horizontal pipes, the equivalent static force will be considered to act concurrently with the full dead load of the pipe, including contents.

(c) Connections. All connections and brackets for pipe will be designed to resist concurrent dead and equivalent static forces. The seismic forces will be determined from the appropriate provisions below. Supports will be provided at all pipe joints unless continuity is maintained. See figure 10-8 for acceptable sway bracing details.

(d) Flexible couplings and expansion joints. Flexible couplings will be provided at the bottoms of risers for pipes larger than 3-1/2 inches in diameter. Flexible couplings and expansion joints will be braced laterally unless such lateral bracing will interfere with the action of the flexible coupling or

expansion joint. See figure 10-9 for typical details of pipe entrance to buildings. See figures 12-4 and 12-7 (chap 12, Utility Systems) for some typical flexible couplings.

(e) Spreaders. Spreaders will be provided at appropriate intervals to separate adjacent pipe lines unless the pipe spans and the clear distance between pipes are sufficient to prevent contact between the pipes during an earthquake.

(2) *Rigid and rigidly attached piping systems.* Rigid and rigidly attached pipes will be designed in accordance with paragraph 10-3. The equivalent static lateral force is given by the formula (3-8) in chapter 3, paragraph 3-3(G),

$$F_p = Z I C_p W_p \qquad (3\text{-}8)$$

where C_p is equal to 0.30, and W_p is the weight of the pipes, the contents of the pipes, and attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. A piping system is assumed rigid if the maximum period of vibration is 0.05 second (for pipes that are not rigid see para (3) below). Figures 10-4, 10-5, and 10-6, which are based on water-filled pipes with periods equal to 0.05 second, are to be used to determine the allowable span-diameter relationship for Zones 1, 2, 3, and 4 for standard (40S) pipe; extra strong (80S) pipe; Types K, L, and M copper tubing; and 85 red brass or SPS copper pipe.

(3) *Flexible piping systems.* Piping systems that are not in accordance with the rigidity requirements of paragraph 10-7c(2) (i.e., period less than 0.05 seconds) will be considered to be flexible (i.e., period greater than 0.05 seconds). Flexible piping systems will be designed for seismic forces with consideration given to both the dynamic properties of the piping system and the building or structure in which it is placed. In lieu of a more detailed analysis, the equivalent static lateral force is given by formula 10-2 of paragraph 10-4e,

$$F_p = Z I A_p C_p W_p \qquad (10\text{-}2)$$

where $A_p = 5.0$, $C_p = 0.30$, and W_p is the weight of the pipes, the contents of the pipes, and attachments. The forces will be distributed in proportion to the weight of the pipes, contents, and attachments. Figure 10-7 may be used to determine maximum spans between lateral supports for flexible piping systems. The values are based on Zone 4 water-filled pipes with no attachments. If the weight of the attachments is greater than 10 percent of the weight of the pipe, the attachments will be separately braced or substantiating calculations are required. Temperature stresses have not been considered in figure 10-7. If temperature stresses ar appreciable, substantiating calculations are quired.

(a) Use of Figure 10-7. The maximum spans and design forces were developed for $Z\ I\ A_p\ C_p = 1.50$. For lower $Z\ I\ A_p\ C_p$ values the spans and forces may be adjusted by the values in table 10-2.

Table 10-2. Multiplicction Factors for Figure 10-7, in Seismic Zones 1, 2, and 3 or When ZIA_pC_p Not Equal to 1.5

Zone	L (feet)	F (pounds)	ZIA_pC_p
3	1.1	0.8	1.12
2	1.25	0.5	0.56
1	1.35	0.3	0.28

(b) Separation between pipes. Separation will be a minimum of four times the calculated maximum displacement due to F_p, but not less than 4 inches clear between parallel pipes, unless spreaders are provided (para 10-7c(1)(e)).

(c) Clearance from walls or rigid elements will be a minimum of three times the calculated displacement due to F_p, but not less than 3 inches clear from rigid elements.

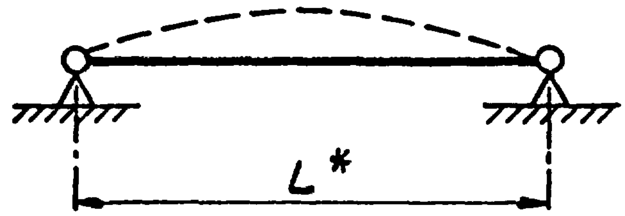
(4) *Alternative method for flexible piping systems.* If the provisions in the above paragraphs appear to be too severe for an economical design, alternative methods based on the rationale describ in paragraph 10-4, *Flexible and Flexibly Moun Equipment,* and paragraph 10-8, *Stacks in Buildings,* may be applied to flexible piping systems.

10-8. Stacks. Stacks are actually beams with distributed mass and, as such, cannot be approximated accurately by single-mass systems. The design criteria presented herein apply to either cantilever or singly-guyed stacks. All stacks designed under the provisions of this paragraph must have a constant moment of inertia or must be approximated as having a constant moment of inertia. Stacks having a slightly varying moment of inertia will be treated as having a uniform moment of inertia with a value equal to the average moment of inertia.

a. Stacks on Buildings. Stacks that extend more than 15 feet above a rigid attachment to the building will be designed according to the criteria prescribed below. Stacks that extend less than 15 feet will be designed for the forces prescribed in chapter 3, paragraph 3-3(G), table 3-4, with $C_p = 0.80$.

(1) *Cantilever stacks*

(a) The fundamental period of the stack be determined from the period coefficient (i.e., C



DIAMETER INCHES	STD.WT. STEEL PIPE 40 S	EX.STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	6'-6"	6'-6"	5'-0"	4'-9"	4'-6"	5'-6"
1½	7'-6"	7'-9"	5'-9"	5'-6"	5'-6"	6'-6"
2	8'-6"	8'-6"	6'-6"	6'-6"	6'-3"	7'-0"
2½	9'-3"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
3	10'-3"	10'-6"	7'-9"	7'-6"	7'-6"	8'-9"
3½	11'-0"	11'-0"	8'-3"	8'-3"	8'-0"	9'-3"
4	11'-6"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
5	12'-9"	13'-0"	10'-0"	9'-6"	9'-6"	10'-9"
6	13'-9"	14'-0"	10'-9"	10'-6"	10'-3"	11'-6"
8	15'-6"	16'-0"				
10	17'-0"	17'-6"				
12	18'-3"	19'-0"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

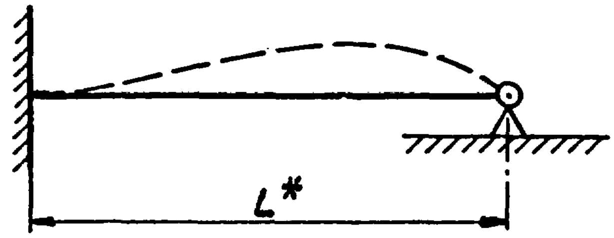
$L^2 = 0.50\pi T_a \sqrt{EIg/w}$

E = MODULUS OF ELASTICITY OF PIPE

I = MOMENT OF INERTIA OF PIPE

w = WEIGHT PER UNIT LENGTH OF PIPE AND WATER

Figure 10-4. Maximum Span for Rigid Pipe Pinned-Pinned



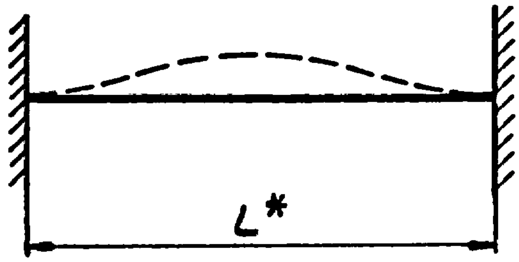
DIAMETER INCHES	STD. WT. STEEL PIPE 40 S	EX. STRONG STEEL PIPE 80 S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	8'-0"	8'-0"	6'-0"	6'-0"	5'-9"	6'-9"
1½	9'-6"	9'-6"	7'-3"	7'-0"	7'-0"	8'-0"
2	10'-6"	10'-9"	8'-0"	8'-0"	8'-9"	9'-0"
2½	11'-9"	11'-9"	9'-0"	8'-9"	8'-6"	9'-9"
3	12'-9"	13'-0"	9'-9"	9'-6"	9'-3"	10'-9"
3½	13'-6"	14'-0"	10'-6"	10'-3"	10'-0"	11'-6"
4	14'-6"	14'-9"	11'-0"	11'-0"	10'-9"	12'-3"
5	16'-0"	16'-3"	12'-3"	12'-0"	11'-9"	13'-3"
6	17'-0"	17'-9"	13'-6"	13'-0"	12'-9"	14'-3"
8	19'-3"	20'-0"				
10	21'-3"	22'-0"				
12	23'-0"	23'-6"				

*MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

$$L^2 = 0.78\pi T \sqrt{EIg/w}$$

SEE FIGURE 10-4 FOR NOTATIONS

Figure 10-5. Maximum Span for Rigid Pipe Fixed-Pinned



DIAMETER INCHES	STD. WT. STEEL PIPE 40S	EX. STRONG STEEL PIPE 80S	COPPER TUBE TYPE K	COPPER TUBE TYPE L	COPPER TUBE TYPE M	85 RED BRASS & SPS COPPER PIPE
1	9'-6"	9'-6"	7'-3"	7'-3"	7'-0"	8'-0"
1½	11'-6"	11'-6"	8'-6"	8'-6"	8'-3"	9'-9"
2	12'-9"	13'-0"	9'-9"	9'-6"	9'-6"	10'-9"
2½	14'-0"	14'-3"	10'-9"	10'-6"	10'-6"	11'-9"
3	15'-6"	15'-9"	11'-9"	11'-6"	11'-3"	13'-0"
3½	16'-6"	16'-9"	12'-6"	12'-3"	12'-0"	14'-0"
4	17'-3"	17'-9"	13'-6"	13'-0"	13'-0"	14'-9"
5	19'-0"	19'-6"	15'-0"	14'-6"	14'-3"	16'-0"
6	20'-9"	21'-3"	16'-3"	15'-9"	15'-6"	17'-3"
8	23'-3"	24'-3"				
10	25'-9"	26'-6"				
12	27'-6"	28'-6"				

* MAXIMUM UNSUPPORTED OR UNBRACED LENGTHS (L) ARE BASED ON WATER-FILLED PIPES WITH PERIOD (T_a) EQUAL TO 0.05 SEC. WHERE

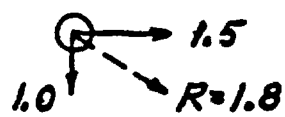
$$L^2 = 1.125\pi T_a \sqrt{EIg/w}$$

SEE FIGURE 10-4 FOR NOTATIONS

Figure 10-6. Maximum Span for Rigid Pipe Fixed-Fixed

Diameter (in.)	Std. Wgt. Steel Pipe - 40S		Ex. Strong Steel Pipe - 80S		Copper Tube Type L	
	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)	L*(ft)	F†(lbs)
1	22	70	22	80	11	17
1-1/2	25	140	26	180	12	35
2	29	220	30	290	14	70
2-1/2	32	380	33	460	15	110
3	34	550	35	710	17	150
3-1/2	36	730	38	930	18	220
4	39	960	40	1,200	19	300
5	41	1,440	44	1,900	20	470
6	45	2,120	46	2,750	22	730
8	49	3,740	54	5,150	26	1,550
10	54	6,080	59	7,670	28	2,620
12	58	8,560	61	10,350	31	3,950

*Maximum spans (L) between lateral supports of flexible piping are based on the resultant of an assumed loading of 1.5 w ($ZIA_pC_p = 1.5$) in the horizontal direction and 1.0 w (gravity) in the vertical direction. The resultant is 1.8 w.



The assumed maximum stress is 20,000 p.s.i. for steel and 7,000 p.s.i. for copper. Simple spans (pinned-pinned) are assumed. The calculated maximum lateral displacements are 3.5 inches for steel ($E = 29 \times 10^6$ p.s.i.) and 0.6 inch for copper ($E = 15 \times 10^6$ p.s.i.).

†The horizontal force (F) on the brace is based on 1.5 w L for the maximum span. For shorter spans, $F_{design} = (L_{design}/L)F$.

Figure 10-7. Maximum Span for Flexible Pipes in Seismic Zone 4
(See Table 10-2 for Other Seismic Zones)

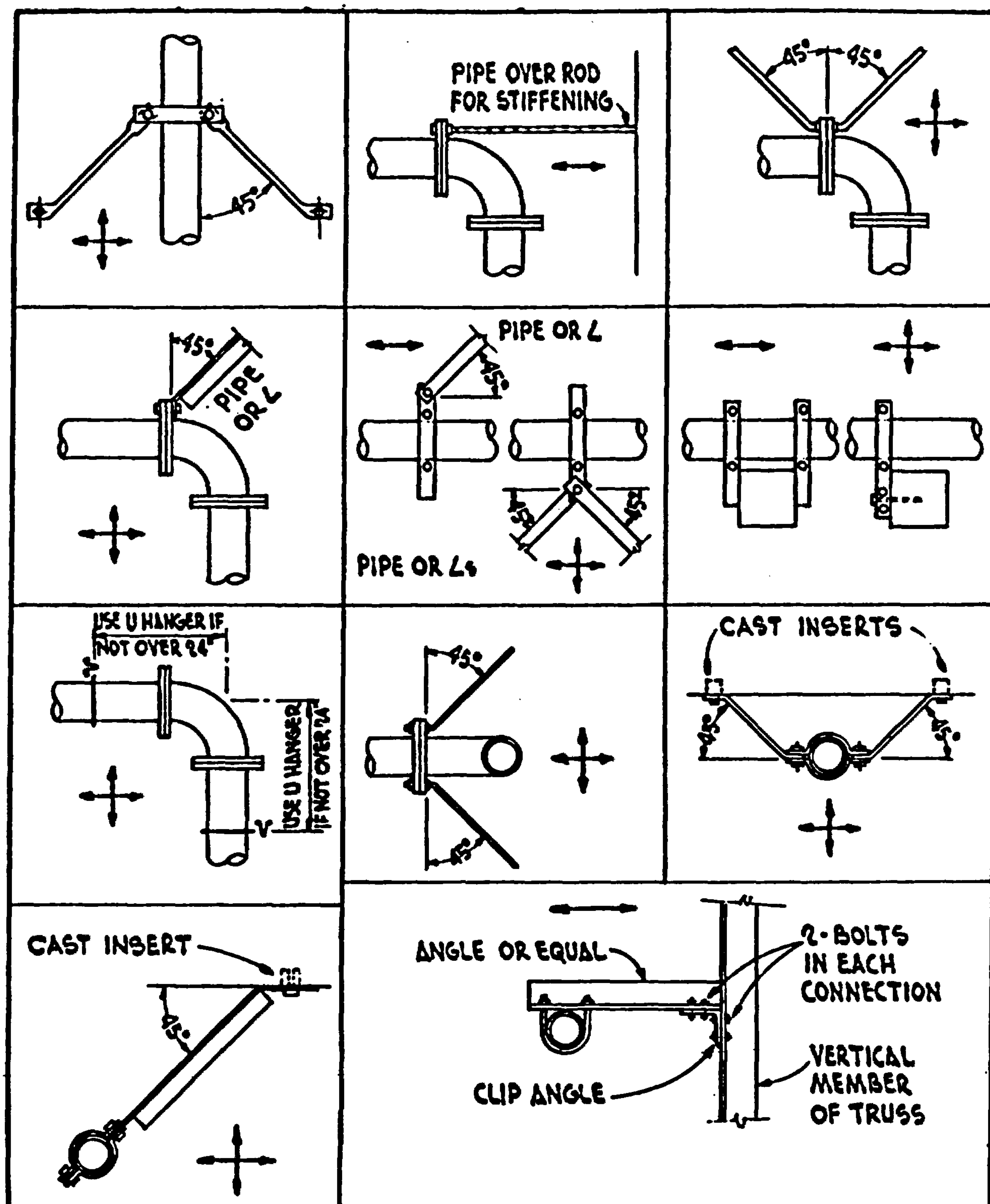


Figure 10-8. Acceptable Seismic Details for Sway Bracing

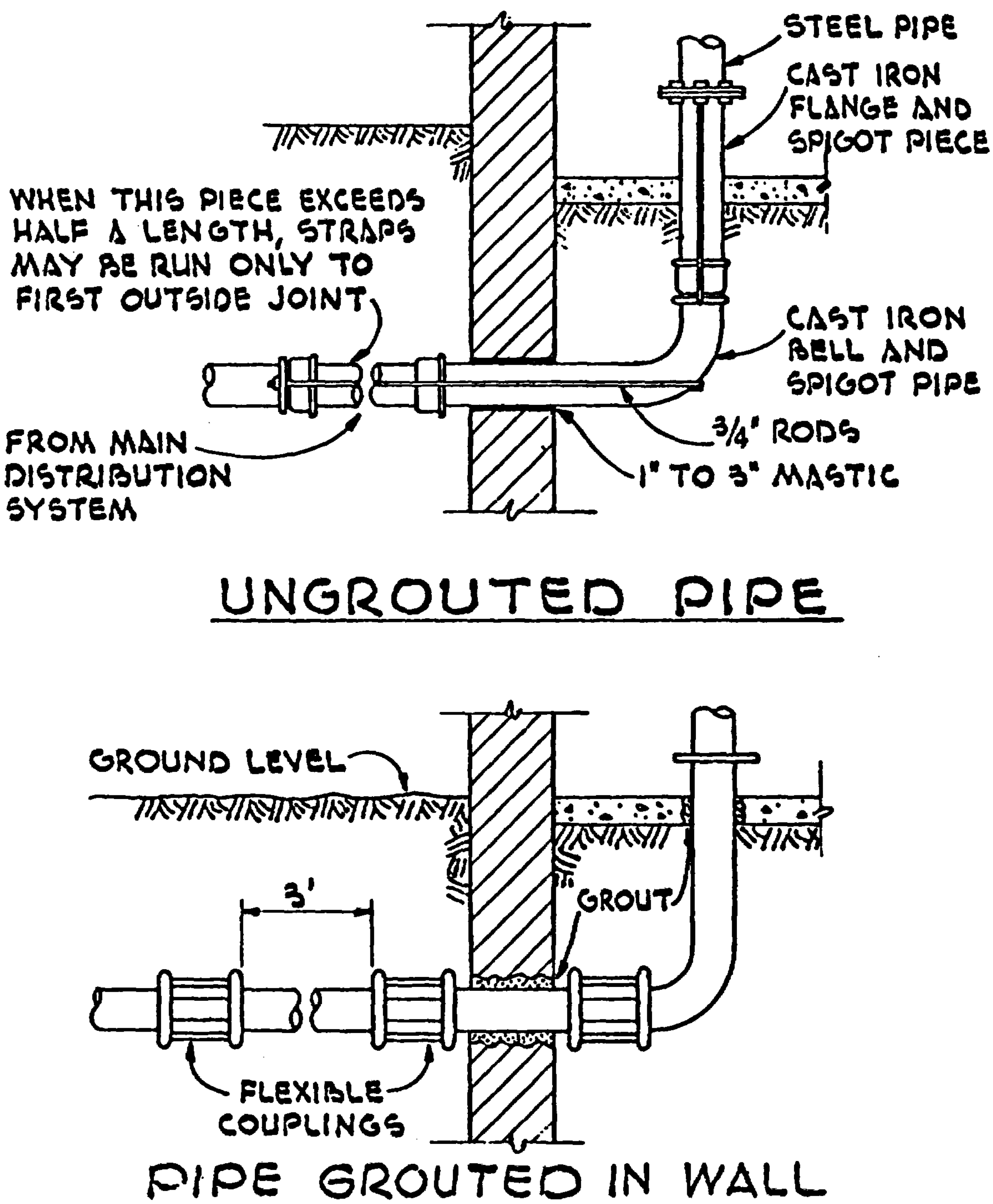


Figure 10-9. Acceptable Seismic Details for Piping Entering Building

0.0909) provided on figure 10-10 unless actually computed.

(b) The equivalent static force will be distributed as an inverted triangle per unit length as shown on figure 10-11.

(c) The static force per unit length at the top of the stack will be determined from the following:

$$f = 1.6\,Z\,I\,A_p\,C_p\,w \qquad (10\text{-}5)$$

where

Z and I are defined in chapter 3

C_p = 0.30 for rigid stacks in Table 3-4

A_p = Amplification factor for coefficient C_p, determined in accordance with paragraph 10-4e

w = Weight per unit length of stack

In no case will the product of A_pC_p be less than 0.8.

(d) If T_a is greater than 0.7 second, an additional concentrated force F_t will be applied to the top of the stack. F_t will be determined by Formula 3-6, where T_a is used in lieu of T and V is the sum of the static forces in paragraph (b). The product of 0.07T need not exceed 0.25.

$$F_t = 0.07TV \qquad (3\text{-}6)$$
$$= 0.07T_a \Sigma f < 0.25 \Sigma f$$

(2) *Guyed Stacks.* The analysis of a guyed stack depends on the relative rigidities of the cantilever resistance and the guy wire support systems. If the wires are very flexible, the stack will respond in the manner of the fundamental mode of vibration of a cantilever (para (1) above). If the wires are very rigid, the stack will respond in a manner similar to the higher modes of vibration of a cantilever with periods and mode shapes similar to those shown in figure 10-10. The fundamental period of vibration of the guyed system will be somewhere between the values for the fundamental and the appropriate higher mode of a similar cantilever stack. An illustration for a single-guyed stack is shown in figure 10-12. The design of guyed stacks is beyond the scope of this manual.

b. Stacks on the Ground. For stacks where the stack foundations are in contact with the ground and the stack is not supported by the building, formula 10-6 will be used in lieu of formula 10-5.

$$f = 1.6\,Z\,I\,(2\,CS)\,w \qquad (10\text{-}6)$$

where C and S are defined in chapter 3. The product of 2 CS will not be less than 0.20. In the loading diagram of figure 10-11, 2 CS will be substituted for the coefficients A_p C_p. If the period of the stack is greater than 0.7 seconds, the additional concentrated force F_t will be applied in accordance with paragraph 10-8a(1)(d).

c. Anchor Bolts. Anchor bolts for moment-resisting stack bases should be as long as possible. A great deal more strain energy can be absorbed with long anchor bolts than with short ones. The use of these long anchor bolts has been demonstrated to give stacks better earthquake performance. In some cases, a pipe sleeve is used in the upper portion of the anchor bolt to assure a length of unbonded bolt for strain energy absorption. When this type of detail is used, provisions will be made for shear transfer (e.g., shear keys, etc.). The use of two nuts on anchor bolts is also recommended to provide an additional factor of safety.

10-9. Bridge cranes and monorails. In addition to the normal horizontal loads prescribed by the various other applicable government criteria, the design of bridge cranes and monorails will also include an investigation of lateral seismic force as set forth in this paragraph.

a. Equivalent Static Force. A lateral force equal to Z C_p times the weight of the bridge crane or monorail will be statically applied at the center of gravity of the equipment. This equivalent static force will be considered to be applied in any direction. C_p will be equal to 0.60.

b. Weight of Equipment. The weight of such equipment need not include any live load, and the equivalent static force so computed will be assumed to act nonconcurrently with other prescribed nonseismic horizontal forces when considering the design of the crane and monorails. When considering the design of the building, the weight of equipment will be included with the weight of the building.

10-10. Elevators. Power-cable driven elevators and hydraulic elevators with lifts over 5 feet will be designed for lateral forces set forth in this chapter.

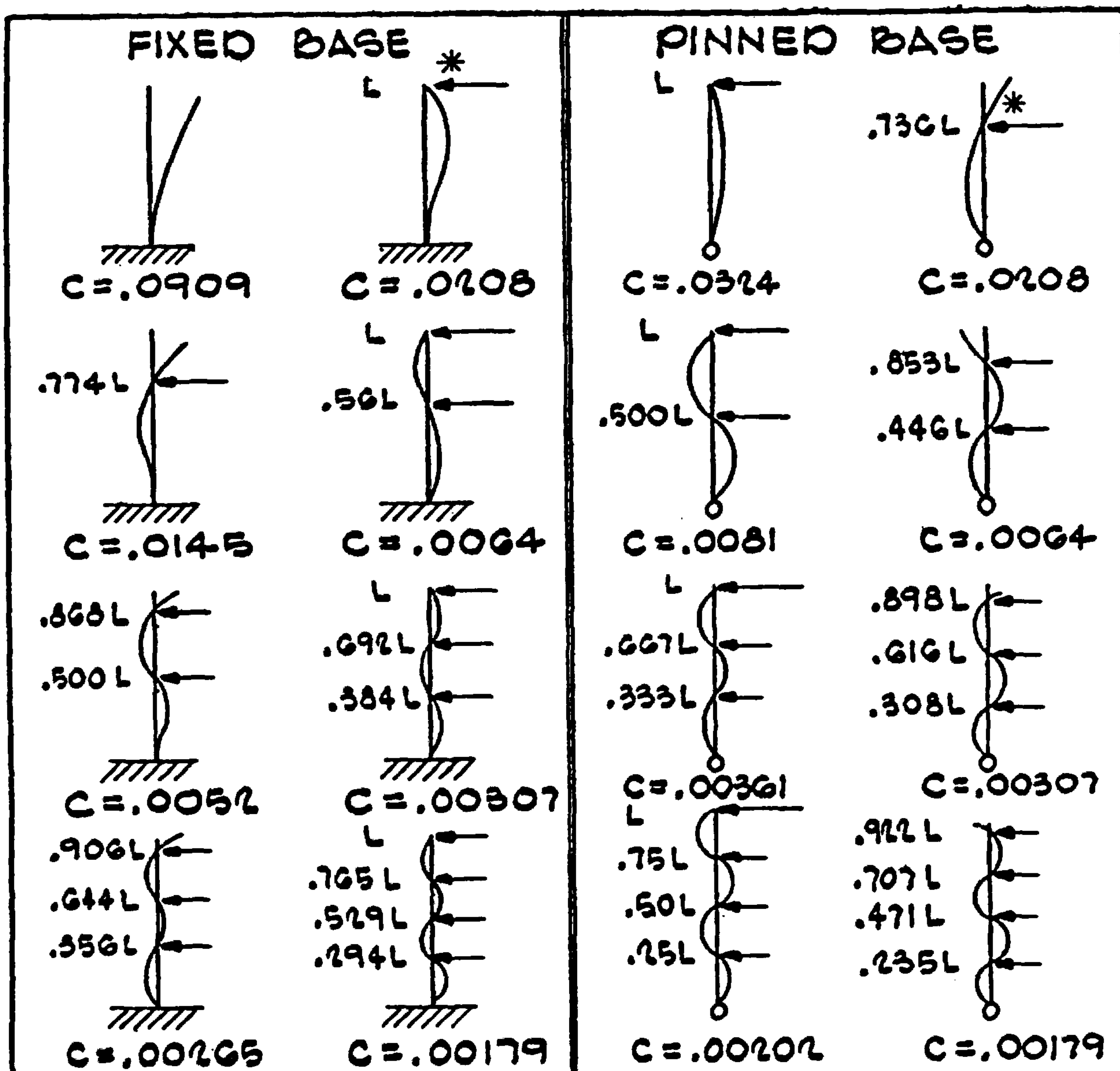
a. Elements of the Elevator Support System. All elements that are part of the elevator support system, such as the car and counterweight frames, guides, guide rails, supporting brackets and framing, driving machinery, operating devices, and control equipment, will be investigated for the prescribed lateral seismic forces. See figure 10-13.

b. Equivalent Static Forces. The lateral seismic forces will conform to the applicable provisions of paragraphs 10-3 and 10-4 and chapter 3, paragraph 3-3(G).

(1) The car and counterweight frames, roller guide assembly, retainer plates, guide rails, and supporting brackets and framing will be designed for C_p = 0.30 in Formula 3-8

$$F_p = Z\,I\,C_p\,W_p \qquad (3\text{-}8)$$

where W_p for the elevator cars is the weight of the



$$T_a = C\sqrt{\frac{wL^4}{EI}}$$

T_a = FUNDAMENTAL PERIOD (SEC.)
w = WEIGHT PER UNIT LENGTH OF BEAM (LB/IN)
L = TOTAL BEAM LENGTH (IN.)
I = MOMENT OF INERTIA (IN.^{4})
E = MODULUS OF ELASTICITY (PSI)
C = PERIOD CONSTANT
* ARROWS DENOTE NODAL POINTS OR POINTS OF NO DISPLACEMENT

Figure 10-10. Period Coefficients for Uniform Beams

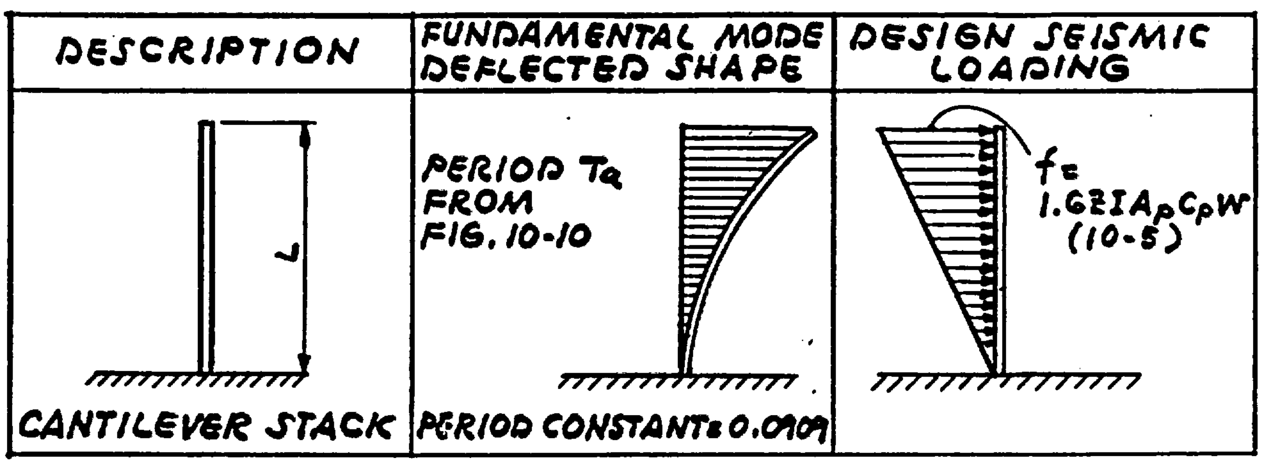


Figure 10-11. Seismic Loading on Cantilever Stack

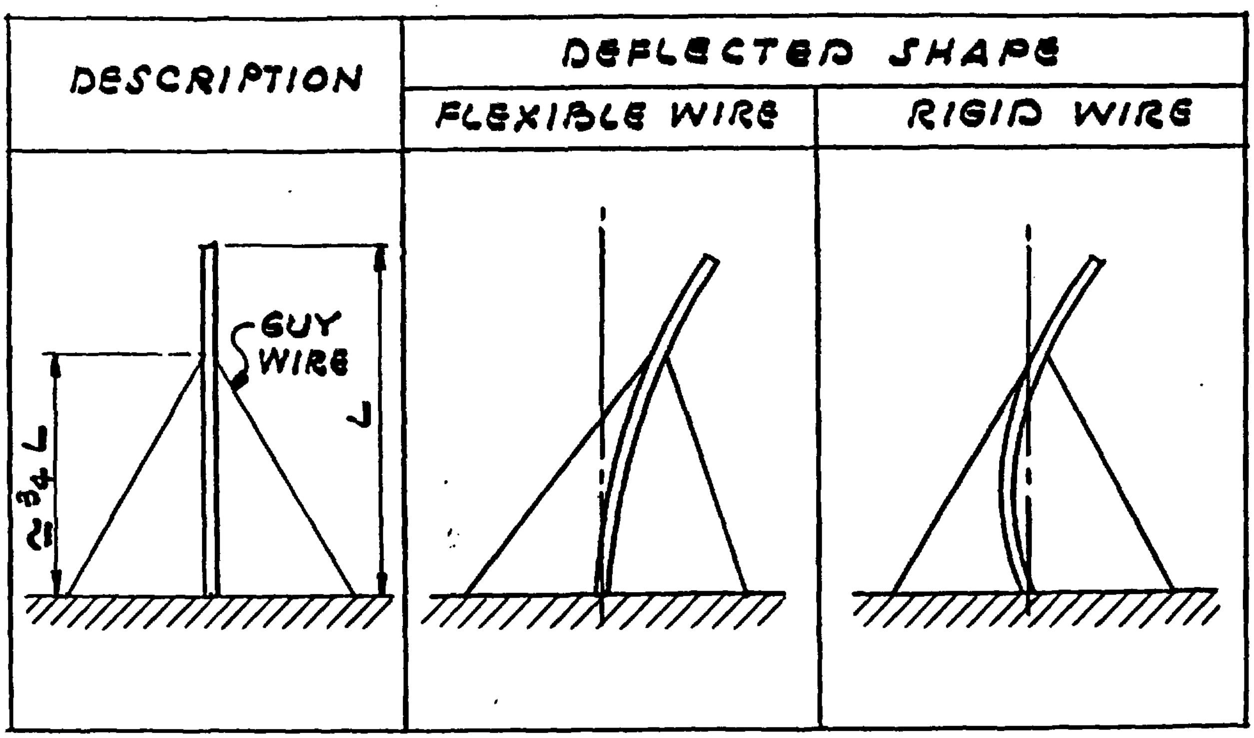
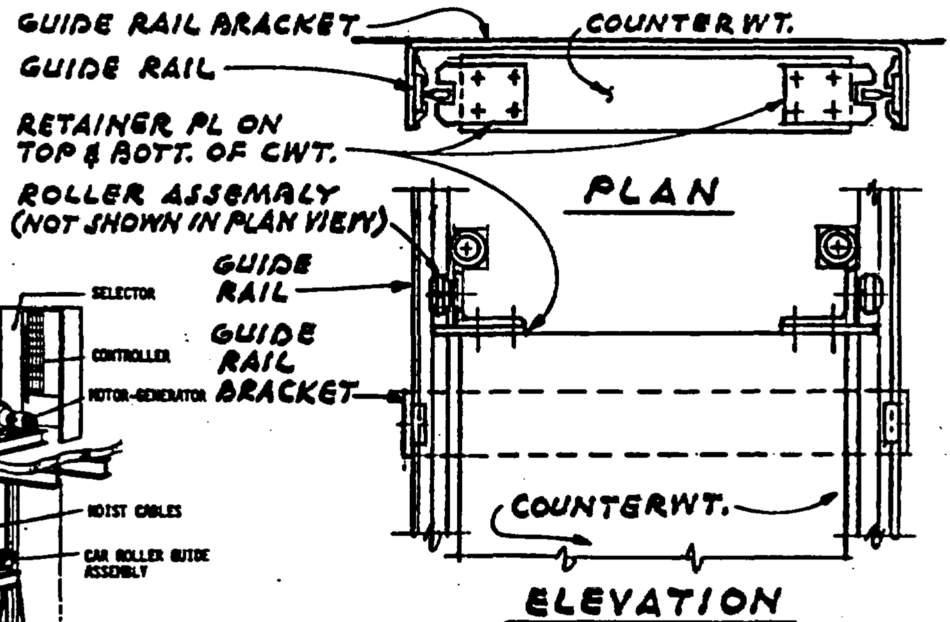
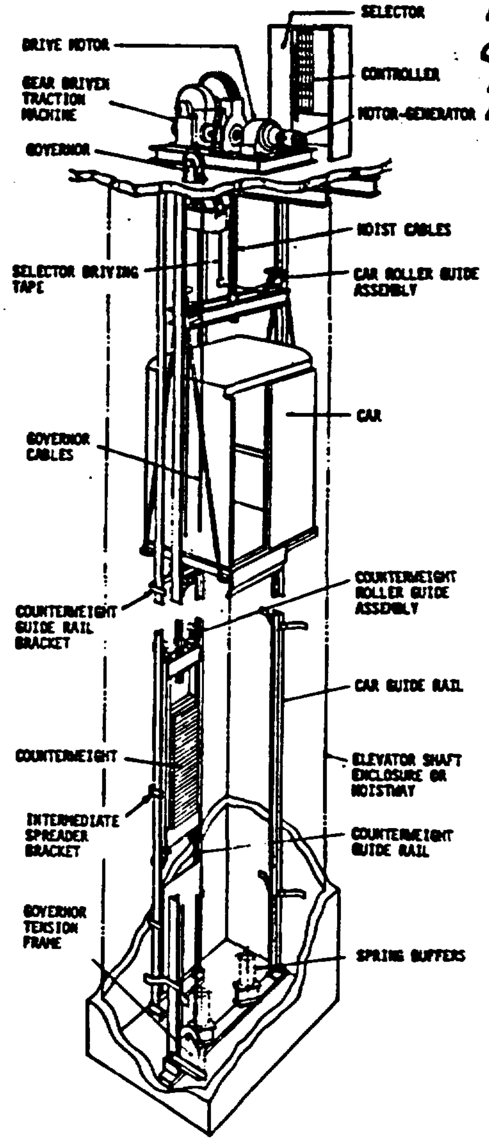
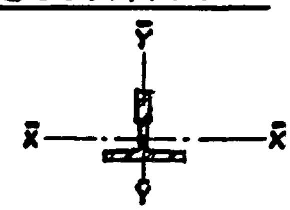


Figure 10-12. Single-Guyed Stack





TRACTION TYPE ELEVATOR



SECTION PROPERTIES OF ELEVATOR GUIDE RAILS

Nom. Wt. Per Foot	A	$\bar{y}$	I_x	S_x	r_x	I_y	S_y	r_y
8	2.40	.81	1.40	.86	.78	1.28	.73	.74
11	3.24	1.12	4.29	1.80	1.15	2.67	1.19	.90
12	3.51	1.08	4.49	1.85	1.13	3.61	1.45	1.01
15	4.41	.97	4.78	1.89	1.04	5.51	2.21	1.12
18.5	5.44	1.26	9.67	3.23	1.33	7.45	2.71	1.17
22.5	6.73	1.38	11.17	4.20	1.28	8.64	3.10	1.13
30	8.93	1.74	24.00	7.37	1.64	11.08	4.03	1.11

A = Area (in.^{2})
$\bar{y}$ = Distance from back of rail to axis x-x (in.)
I_x = Moment of inertia about axis x-x (in.^{4})
S_x = Section modulus about axis x-x (in.^{3})
r_x = Radius of gyration about axis x-x (in.)
I_y = Moment of inertia about axis y-y (in.^{4})
S_y = Section modulus about axis y-y (in.^{3})
r_y = Radius of gyration about axis y-y (in.)

Figure 10-13. Elevator Details

car plus 0.4 times its rated load. The lateral forces acting on the guide rails will be assumed to be distributed 1/3 to the top guide rollers and 2/3 to the bottom guide rollers of elevator cars and counterweights. The elevator car and/or counterweight will be assumed to be located at its most adverse position in relation to the guide rails and support brackets. Horizontal deflections of guide rails will not exceed 1/2 inch between supports and horizontal deflections of the brackets will not exceed 1/4 inch.

(a) In Seismic Zones 3 and 4, a retainer plate (auxiliary guide plate) will be provided at top and bottom of both car and counterweight. The clearances between the machined faces of the rail and the retainer plate shall not be more than 3/16 inch and the engagement of the rail shall not be less than the dimension of the machined side face of the rail. When a car safety device attached to the lower members of the car frame comply with the lateral restraint requirements, a retainer plate is not required for the bottom of the car.

(b) In Seismic Zones 3 and 4, the maximum spacing of the counterweight rail tie brackets tied to the building structure shall not exceed 16 feet. An intermediate spreader bracket, not required to be tied to the building structure, shall be provided for tie brackets spaced greater than 10 feet and two intermediate spreader brackets are required for tie brackets greater than 14 feet.

(2) Machinery and equipment will be designed for C_p = 0.30 in Formula 3-8 when rigid and rigidly attached. Flexible or flexibly mounted equipment will be designed in accordance with paragraph 10-4.

10-11 Typical details for securing equipment. See figures 10-14 and 10-15 for examples of seismic restraints for equipment.

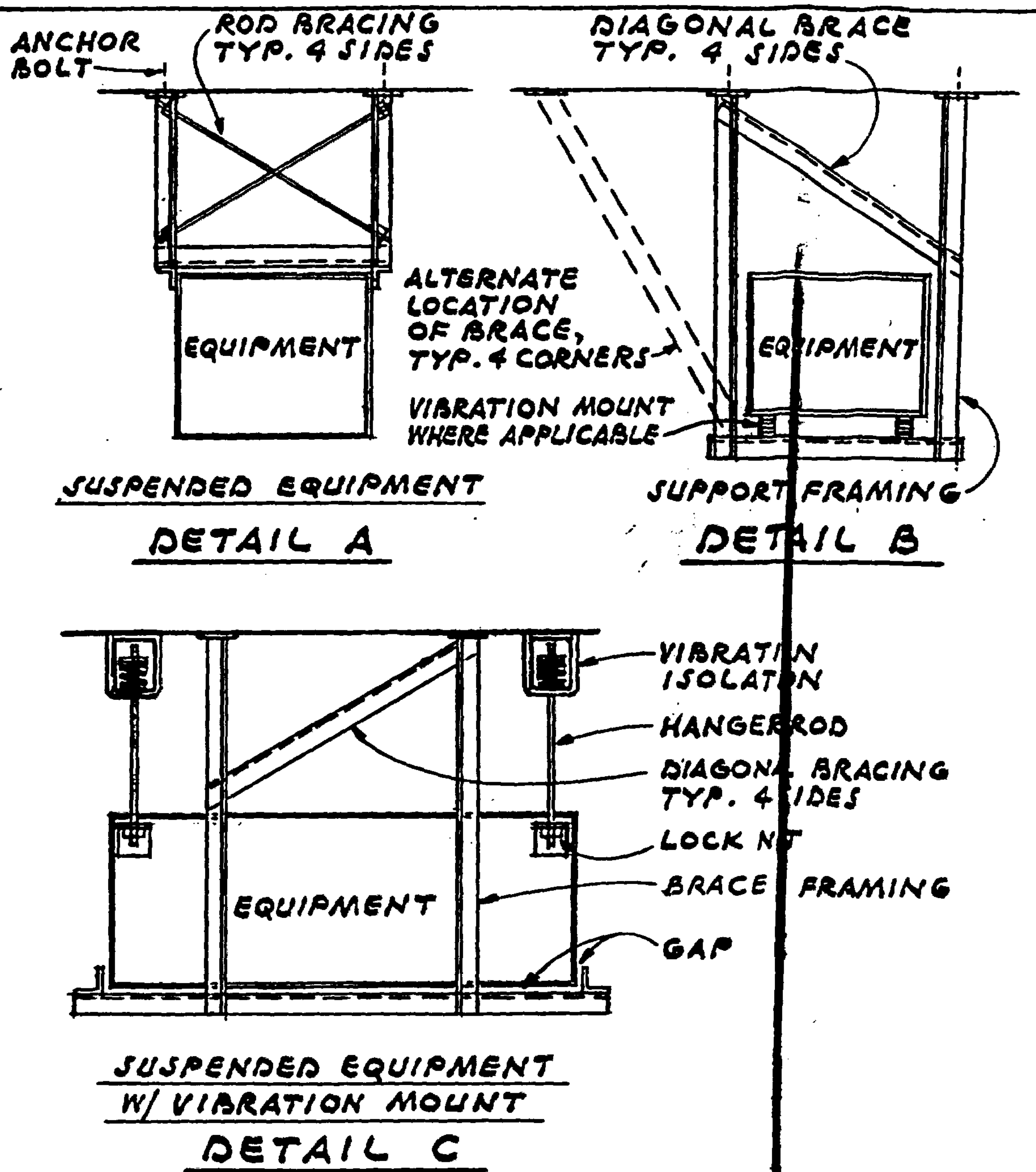


Figure 10-14. Typical Seismic Restraint of Mechanical Equipment

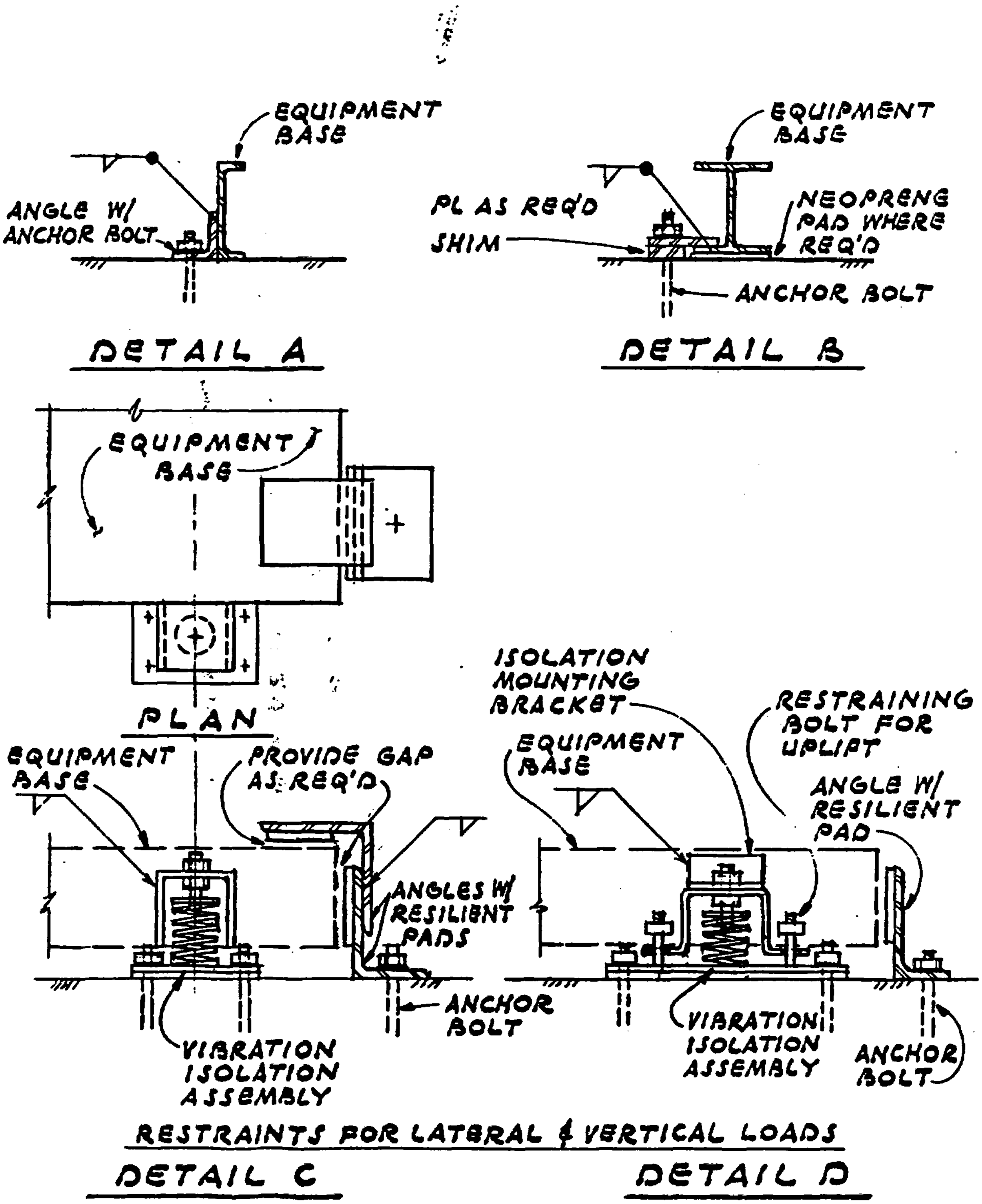


Figure 10-15. Typical Seismic Restraint of Mechanical Equipment

CHAPTER 11
STRUCTURES OTHER THAN BUILDINGS

11-1. Purpose and scope. This chapter prescribes the seismic design criteria for structures other than buildings (e.g., chap 3, table 3-3, categories 7 and 8). This includes structures, independent of buildings, that are located on the ground. Refer to chapter 10, Mechanical and Electrical Elements, for seismic design criteria for equipment. In some cases, equipment qualifies under this chapter (chap 10, para 10-5c). For stacks on the ground refer to chapter 10, paragraph 10-8b.

11-2. General requirements. Structures other than buildings are designed in accordance with chapter 3, paragraph 3-3D, formula 3-1

$$V = ZIKCSW \tag{3-1}$$

where K is equal to 2.5 for certain elevated tanks and inverted pendulums (category 7, table 3-3) and K is equal to 2.0 for other structures (category 8, table 3-3). Structures that have uniformly distributed mass may have the lateral force distributed in a manner similar to cantilever stacks (see chap 10, para 10-8*b* and fig 10-11). Structures that can be approximated by lumped mass systems will have the lateral force distributed in a manner similar to buildings (chap 3, para 3-3(E)). Single degree of freedom systems will have the lateral force applied at the center of gravity of the mass of the structure.

11-3. Elevated tanks and other inverted pendulum structures. Structures that represent inverted pendulums, such as an elevated tank supported by a tower structure that is light in weight relative to the tank and contents, will use the basic formula V = ZIKCSW with the value of K equal to 2.5. The minimum value of KC is 0.12. The value for W will include the effective weight of the contents. The accidental torsion will be computed as for buildings. Stresses will be computed for the earthquake forces in any horizontal direction.

a. Elevated Tanks on Cross-Braced Columns. Foundation piers shall be interconnected by steel or reinforced concrete struts. When supported by piles or caissons, diagonal struts will also be required. For most four-legged tanks, uplift and column design is critical when the horizontal force is applied at 45° to the major axes (see chap 4, para 4-4*c*(1)(b)). Example G-1 in appendix G illustrates the method of obtaining the seismic forces on a four-legged water tank, including a method for computing the period of vibration required to determine the values for the C and S coefficients.

b. Hydrodynamic Effects. In general, W will include the total weight of the contents of an elevated tank. However, properly substantiated procedures that account for the reduction of the effective weight of the liquid due to sloshing may be used. Such procedures usually result in a mathematical model that represents a two-degree-of-freedom system consisting of an effective rigid mass of liquid and an effective sloshing mass of liquid. The procedure is similar to that used for vertical tanks on the ground (para 11-4) and some of the technical publications referenced in paragraph 11-4 are applicable.* In addition to designing the tower to resist the equivalent static seismic forces, the effects of the sloshing liquid on the interior of the tank will be considered.

c. Elevated Tanks, Pedestal Types. Pedestal type elevated water tanks will not be permitted is Seismic Zone Nos. 3 and 4. In Seismic Zone Nos. 1 and 2, K will be equal to 3.0.

11-4. Vertical tanks (on ground). The basic formula V = ZIKCSW will be used for tanks in which the liquid is rigidly contained (i.e., sloshing prevented), for tanks holding highly viscous materials, and for pressure tanks. The value of K is equal to 2.0 (chap 3, table 3-3), W is the weight plus contents, and for calculating C and S the period T will be assumed less than 0.3 seconds unless substantiated to be longer (i.e., CS = 0.133 to 0.140 per table 4-3 in chap 4). For tanks where the liquid is not rigidly contained, the hydrodynamic effects of the sloshing liquid may be considered in order to reduce the effective mass and determine the effective centroid of the liquid.

a. Hydrodynamic Effects. During an earthquake there is a complex redistribution of pressures in a tank. The design procedure for considering these hydrodynamic effects is based on a simplified procedure described and modified in several technical publications.[1-7]** The effective force distribution is illustrated in figure 11-1. The liquid is divided into a constrained portion and an in-motion portion. (If h is less than 1.5R there is no contrained liquid.) Part of the in-motion liquid, combined with the constrained liquid, forms the effective mass of the impulsive force P_I ($P_1 + P_2 = P_I$). The remaining portion of

*References 1-4 in paragraph 11-8.

**References listed in paragraph 11-8.

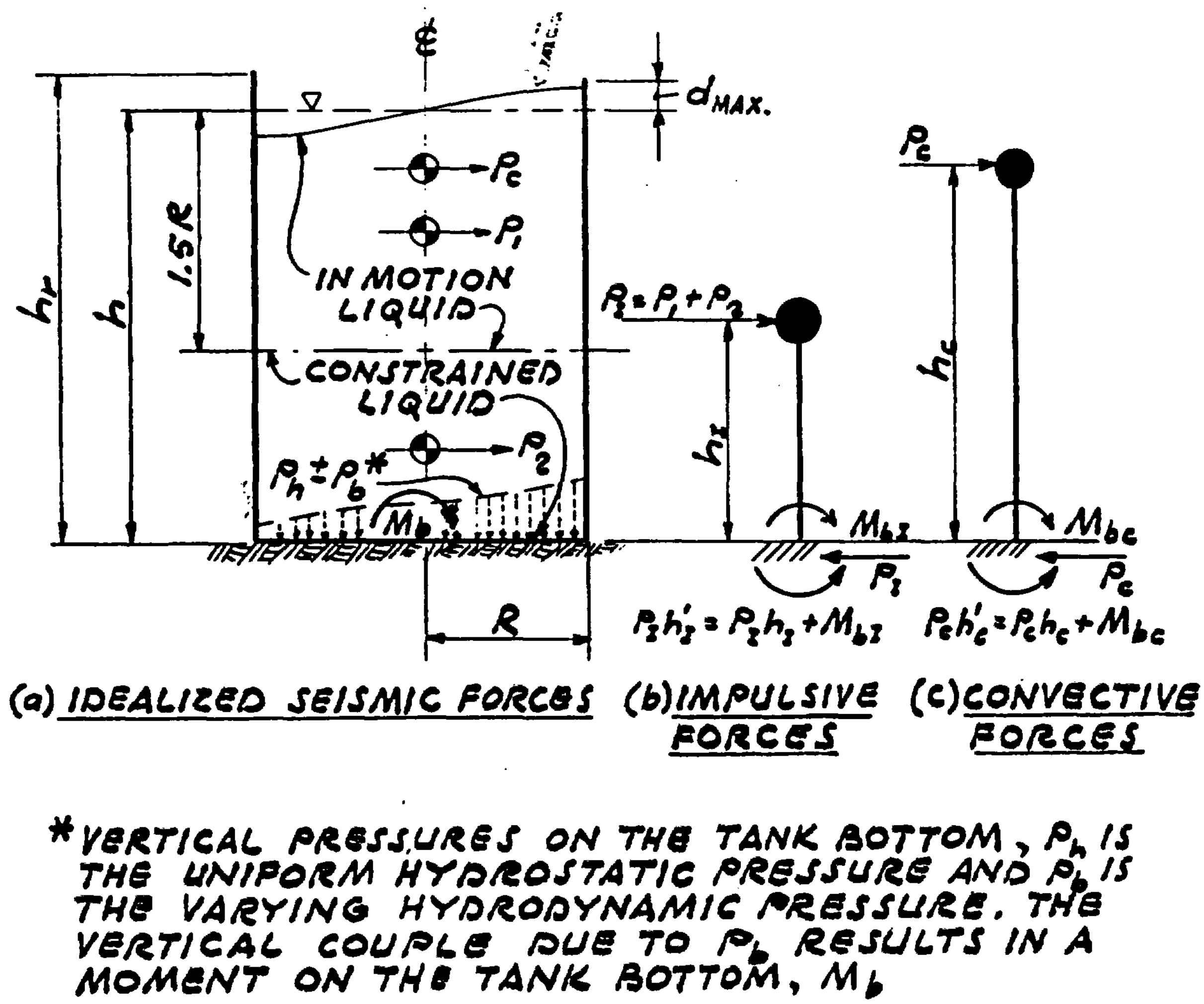


Figure 11-1. Effective Liquid Force Distribution

in-motion liquid forms the mass for the convective force P_C. P_I and P_C are the resultant forces of the horizontal pressures on the sides of the tank. P_I represents the force of the effective mass of liquid that moves rigidly with the tank and P_C represents the force of the effective mass of the sloshing liquid. In addition to P_I and P_C, there is a vertical couple, M_b, acting on the bottom of the tank due to the unbalanced vertical pressures (P_b). Bending and overturning moments are determined by multiplying P_I and P_C by the effective heights h_I and h_c, respectively. In order to include the effects of M_b below the tank base, modified effective heights h'_I and h'_c are given.

(1) *Rigid body forces.* The rigid body forces (fig 11-2a) include the seismic forces due to the impulsive liquid, the walls of the tank and the roof. The term rigid body is used to denote the impulsive liquid moving rigidly with the tank. Actually, the tank does have some flexibility depending on the size and shape. For calculating C and S it will be assumed that the period of the tank and contents is less than 0.3 second unless substantiated to be longer.

(a) The total horizontal rigid body force, V_{RB}, will be determined by formula 11-1,

$$V_{RB} = Z\,I\,K\,C\,S\,(W_r + W_w + W_I) \qquad (11\text{-}1)$$

where Z and I are prescribed in chapter 3, K equals 2.0, and CS equals 0.14 unless a lower value is substantiated. W_r is the weight of the roof (if any), W_w is the weight of the tank walls, and W_I is the weight of the impulsive liquid. W_I is determined from the effective weight ratio, W_I/W, in figure 11-3 or table 11-1, where W is the total weight of the liquid.

(b) The moments at the base of the tank are determined by formula 11-2,

$$M_{RB} = Z\,I\,K\,C\,S\,[W_r h_r + W_w \bar{h}_w + W_I h_I] \qquad (11\text{-}2)$$

where h_r is the height of the roof, $\bar{h}_w$ is the height to the center of mass of the tank walls, and h_I is the effective height of the impulsive liquid. h_I is determined from the effective height ratio, h_I/h, in figure 11-3 or table 11-2, where h is the height of the water level (at rest). To calculate stresses in the tank wall, where M_b is not effective, use h_I. Below the tank base, where M_b is effective, use h_I.

(2) *Sloshing liquid forces (Figure 11-2b).*

(a) The sloshing liquid forces V_{SL} are equal to the convective force, P_C, and will be determined by formula 11-3,

$$V_{SL} = Z\,I\,K\,C\,S\,W_C \qquad (11\text{-}3)$$

where Z, I, and K are the same as used in formula 10-1. C and S are dependent on the sloshing period T (para (b) below) and the site period T_S (refer to chap 3). W_C, the weight of the convective liquid, is determined from the effective weight ratio, W_C/W, in figure 11-3 or table 11-1, where W is the total weight of the liquid.

(b) The sloshing period is determined by formula 11-4,

$$T = k_T\sqrt{h} \qquad (11\text{-}4)$$

where k_T is determined from figure 11-4 or table 11-3.

(c) The moments at the base of the tank are determined by formula 11-5,

$$M_{SL} = Z\,I\,K\,C\,S\,W_c h_c \qquad (11\text{-}5)$$

where h_c is the effective height of the convective liquid. h_c is determined from the effective height ratio, h_c/h, in figure 11-3 or table 11-2, where h is the height of the water level (at rest). To calculate stresses in the tank wall, where M_b is not effective, use h_c. Below the tank base, where M_b is effective, use h'_c.

(d) The maximum design height of the sloshing wave is determined from formula 11-6 for cylindrical tanks

$$d_{max} = \frac{0.75\,(Z\,I\,K\,C\,S)\,R}{1 - k_d(Z\,I\,K\,C\,S)} \qquad (11\text{-}6)$$

and from formula 11-7 for rectangular tanks

$$d_{max} = \frac{0.833\,(Z\,I\,K\,C\,S)\,R}{1 - k_d(Z\,I\,K\,C\,S)} \qquad (11\text{-}7)$$

where k_d is obtained from figure 11-5 or table 11-4. R is the radius of a cylindrical tank or one-half the plan dimension of a rectangular tank.

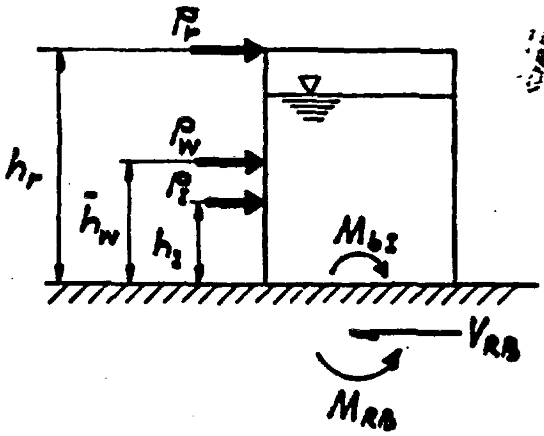
(3) *Combining the rigid body forces and the sloshing liquid forces.* The rigid body forces and the sloshing forces will be combined by the square root of the sum of the squares as shown in formulas 11-8 and 11-9.

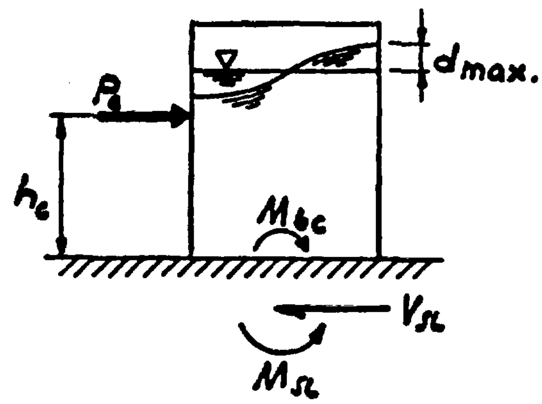
$$V_{total} = \sqrt{V_{RB}^2 + V_{SL}^2} \qquad (11\text{-}8)$$

$$M_{total} = \sqrt{M_{RB}^2 + M_{SL}^2} \qquad (11\text{-}9)$$

This is consistent with modal analysis procedures where spectral responses of the predominant modes are combined in such a manner.

(4) *Sloshing wave height d_{max}.* The value of d_{max} must be less than the freeboard height $(h_r - h)$ for the simplified hydrodynamic procedure to be valid. If d_{max} is greater than $(h_r - h)$, liquid will overflow the top of the tank when there is no roof or will be confined by the roof if a roof exists. When there are interior elements, such as baffles or roof supports, the effects of sloshing liquid on these elements will be considered.





$$P_r = ZIKCSW_r$$
$$P_W = ZIKCSW_W$$
$$P_I = ZIKCSW_I$$

$$V_{RB} = P_r + P_W + P_I$$

M_{RB} (TANK SHELL)
$$= P_r h_r + P_W \bar{h}_W + P_I h_I$$

M_{RB} (BELOW BASE)
$$= P_r h_r + P_W \bar{h}_W + P_I h_I + M_{bI}$$
$$= P_r h_r + P_W \bar{h}_W + P_I h'_I$$

Figure 11-2(a). Rigid Body Forces (paragraph 11-4a(1))

$$P_c = ZIKCSW_c$$

$$V_{SL} = P_c$$

M_{SL}(TANK SHELL) $= P_c h_c$

M_{SL}(BELOW BASE) $= P_c h_c + M_{bc}$
$$= P_c h'_c$$

Figure 11-2(b). Sloshing Liquid Forces (paragraph 11-4a(2))

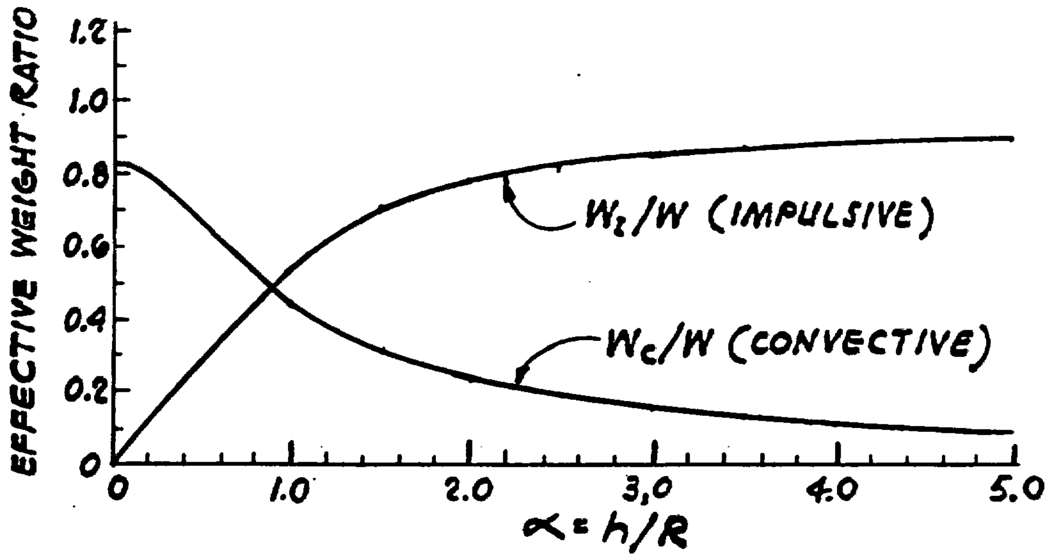


Figure 11-3(a). Effective Weight Ratio (See Table 11-1)

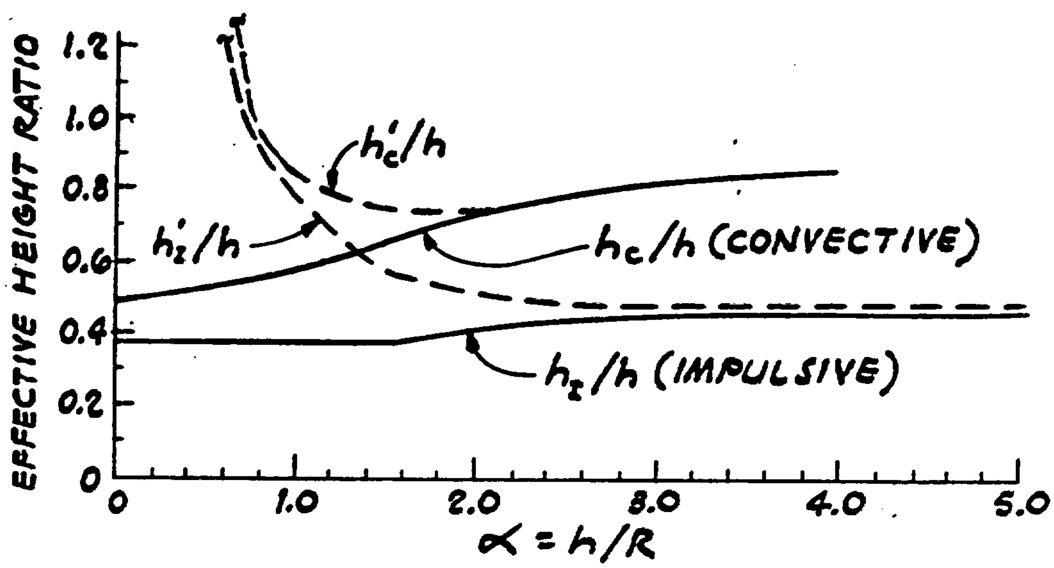


Figure 11-3(b). Effective Height Ratio (See Table 11-2)

Table 11-1. Effective Weight Ratio
(See Figure 11-3(a) for Plot)

α		0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0
W_I/W, impulsive		0.29	0.42	0.54	0.71	0.79	0.83	0.86	0.88	0.89	0.91
W_C/W, convective	Cylindrical	0.66	0.53	0.43	0.30	0.23	0.18	0.15	0.13	0.11	0.09
	Rectangular	0.69	0.58	0.48	0.34	0.26	0.21	0.18	0.15	0.13	0.11

Table 11-2. Effective Height Ratio
(See Figure 11-3(b) for Plot)

α		0.5	0.75	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0
h_I/h, impulsive		0.38	0.38	0.38	0.38	0.41	0.42	0.44	0.45	0.45	0.46
h'_I/h, impulsive		1.6	1.0	0.80	0.58	0.51	0.49	0.48	0.48	0.47	0.47
h_C/h, convective	Cylindrical	0.53	0.57	0.60	0.68	0.74	0.79	0.82	0.84	0.86	0.89
	Rectangular	0.53	0.55	0.58	0.65	0.71	0.76	0.79	0.82	0.84	0.87
h'_C/h, convective	Cylindrical	1.6	0.96	0.79	0.73	0.75	0.79	0.82	0.84	0.86	0.89
	Rectangular	2.0	1.11	0.86	0.73	0.74	0.77	0.80	0.82	0.84	0.87

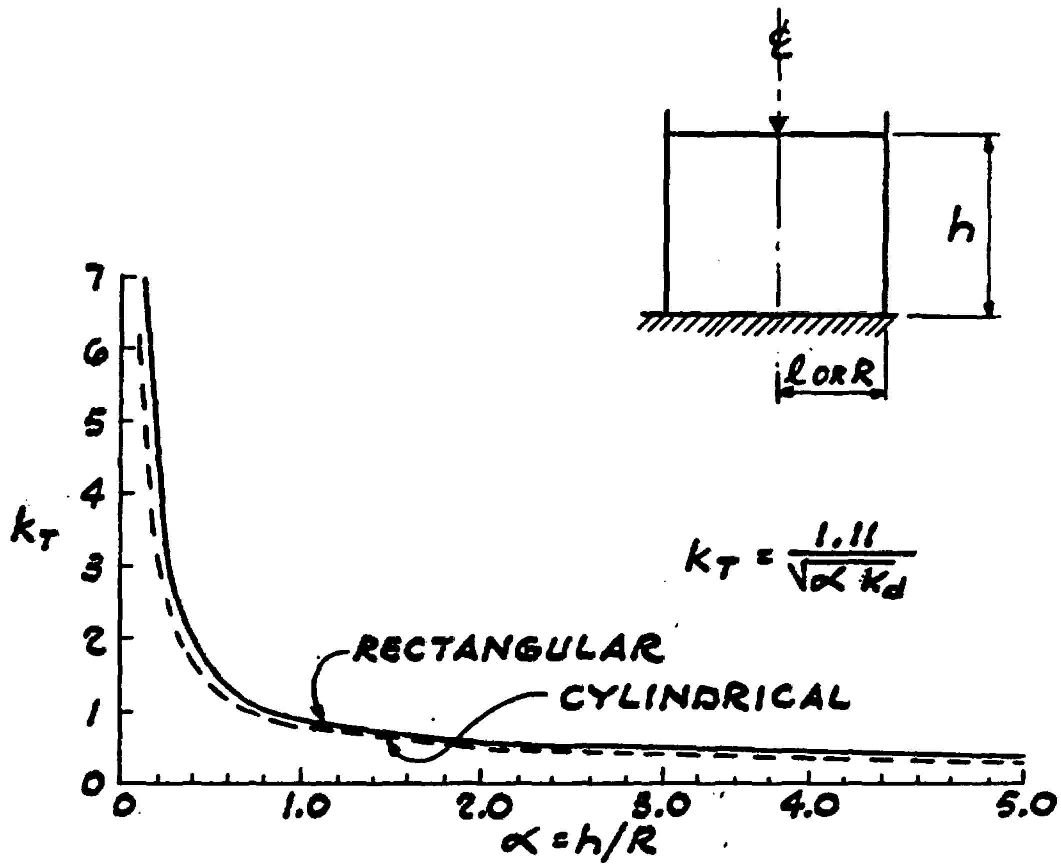


Figure 11-4. Period Constant, k_T (See Table 11-3)

*Table 11-3. Period Constant k_T**
(See Figure 11-4 for Plot)

α	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.0	5.0
k_T, cylindrical	1.4	1.0	0.84	0.67	0.58	0.52	0.47	0.41	0.37
k_T, rectangular	1.5	1.1	0.92	0.73	0.63	0.56	0.51	0.44	0.39

*Sloshing (convective motion) Period, $T = k_T\sqrt{h}$, where h is the height in feet.

Table 11-4. Coefficient k_d
(See Figure 11-5 for Plot)

α	0.5	0.75	1.0	1.5	2.0	2.5	3.0	4.0	5.0
k_d, cylindrical	1.33	1.62	1.75	1.83	1.84	1.84	1.84	1.84	1.84
k_d, rectangular	1.04	1.31	1.45	1.55	1.57	1.58	1.58	1.58	1.58

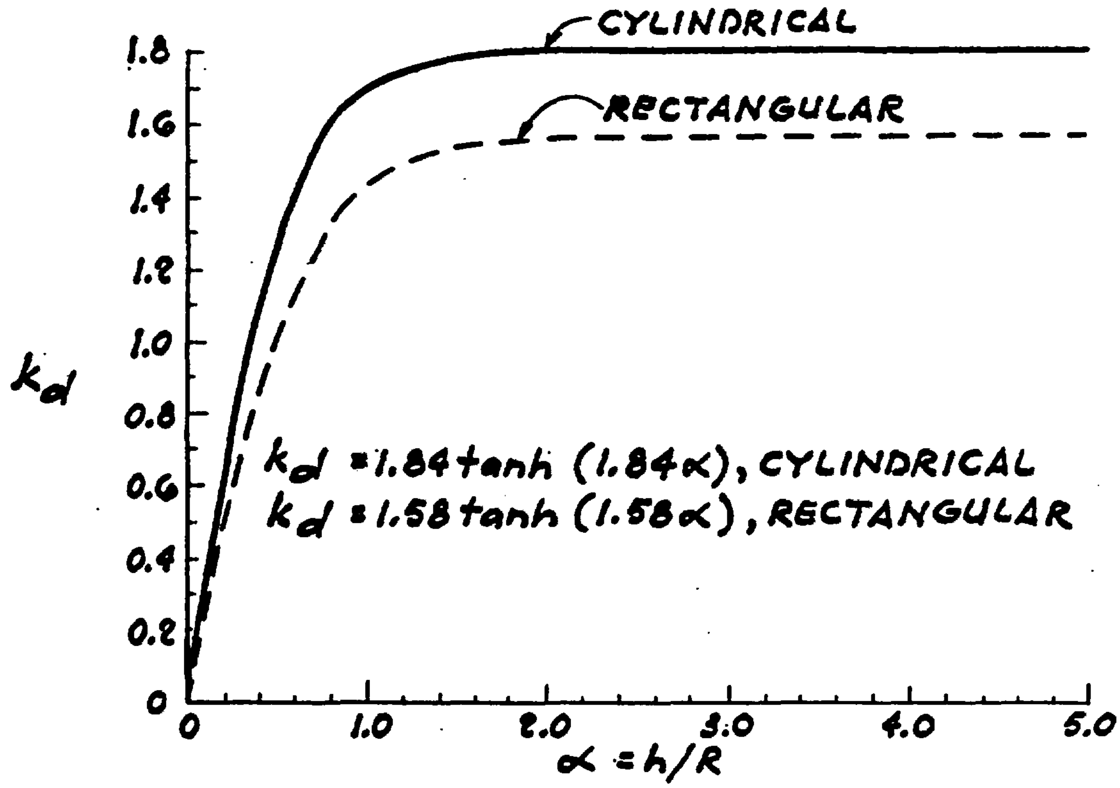


Figure 11-5. Coefficient k_d (See Table 11-4)

b. Design of Tank. The critical items of concern in the seismic design of the tank are (1) horizontal shear at the base, (2) overturning and uplift forces at foundations, (3) compression buckling of the tank shell, and (4) when tie-downs are used, the resulting additional stresses at the attachment of the anchors (e.g., possibility of tearing the shell). The stresses resulting from the seismic forces will be combined with other applicable stresses. Procedures for the design of vertical tanks are beyond the scope of this manual. Industry standards (at the time of this writing) are developing seismic criteria for supplements for the general design criteria [8-10*] (e.g., AWWA and API). Procedures used for the design of tanks will be substantiated by means of rational analysis, tests, or past experience.

11-5. Horizontal tanks (on ground). The basic formula V = ZIKCSW will be used. For this type of tank, the value of K will be 2.0. The critical items of concern in the seismic design are the stresses in the saddles and in the base footing. The soil pressure in the transverse direction due to overturning may be critical. The resultant of forces must always fall within the middle third of the footing pad.

11-6. Retaining walls. The design of retaining walls for seismic forces in Seismic Zone 4 will use an additive seismic factor of 20 percent of the total earth pressure forces plus 20 percent of the weight of the wall at a point 2/3 the fill height above the base of the retaining wall. It is obvious that the stresses in the concrete and reinforcing steel will not be critical as the increase in stresses or decrease in load factor is greater than the increase due to seismic load. The overturning effect on the footing may be critical in some cases. The footing will be sized so that there is no theoretical net tension between footing and the supporting ground. Refer to chapter 4, paragraph 4-8, for design of foundations. In Seismic Zones 1, 2, and 3, the Z factor will be applied to the 20 percent factor used in Seismic Zone 4.

11-7. Buried structures. Buried tanks and pipes of moderate size, or smaller, generally do not require special seismic design considerations if applicable nonseismic design criteria are satisfied. However, tanks, tunnels, pipes, etc., which have large cross-sections, or are classified for critical or important usage, will require special considerations for seismic design that are not included in the scope of this manual. In the design of long structures, consideration will be given to the wave shape resulting from the seismic ground motion. Where changes in the support system, configuration, or soil condition occur, flexible couplings will be provided as discussed in chapter 12.

11-8. References.

a. Housner, G. W., "Dynamic Pressures on Accelerated Fluid Containers," *Bulletin of the Seismological Society of America,* Vol. 47, January 1957, pp. 15-35.

b. U.S. Atomic Energy Commission, "Nuclear Reactors and Earthquakes," TID-7024, Washington, DC, 1963 (corrected 1969), pp. 183-195 and 367-390.

c. U.S. Atomic Energy Commission, "Summary of Current Seismic Design Practice for Nuclear Reactor Facilities," TID-25021, Washington, DC, 1967, pp. 124-137.

d. Blume, J. A., et al., *Earthquake Engineering for Nuclear Reactor Facilities,* John A. Blume & Associates, Engineers, JAB-101, San Francisco, 1971, pp. 111-123.

e. Veletsos, A. S., "Seismic Effects in Flexible Liquid Storage Tanks, "*Proceedings of the Fifth World Conference on Earthquake Engineering,* Rome, Italy, 1973.

f. Veletsos, A. S., and J. Y. Yang, "Earthquake Response of Liquid-Storage Tanks," Advances in Civil Engineering Through Engineering Mechanics, *Proceedings Second Annual Engineering Mechanics Division Specialty Conference,* ASCE, May 1977, pp. 1-24.

g. Clough, D. P., *Experimental Evaluation of Seismic Design Methods for Broad Tanks,* University of California Earthquake Engineering Research Center, Report Number UCB/EERC-77/10, May 1977.

h. American Water Works Association, "AWWA Standard D100 for Welded Steel Tanks for Water Storage" appendix A, Seismic Design of Water Storage Tanks, ANSI/AWWA D100-79, 1979d.

i. American Petroleum Institute, "API Standard 650, Welded Steel Tanks for Oil Storage" Seismic Design of Storage Tanks, 7th Edition—1980, appendix E.

j. Wozniak, R. S., and W. W. Mitchell, "Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks," API, Refining, 43rd midyear meeting, *CBT-5359,* Toronto, Canada, May 1978.

*References in paragraph 11-8.

CHAPTER 12
UTILITY SYSTEMS

12-1. Purpose and scope. This chapter prescribes the criteria for utility systems and components 5 feet or farther beyond buildings in seismic areas. Utility systems have been classified as being either above grade or underground. Principles, factors, and concepts involved in seismic design are illustrated. These are not mandatory, therefore, other equivalent methods or schemes complying with applicable agency guide specifications and the intent of this manual may be used.

12-2. General requirements. Utility systems will be planned and designed in accordance with the provisions given in this chapter, except as follows:

a. Systems Above Grade. Utility system components and equipment supports above grade will be designed in accordance with the applicable provisions of chapter 10, Mechanical and Electrical Elements.

b. Rigorous Analysis. No part of this chapter will be construed to prohibit a rigorous analysis of an exterior utility system either above or below grade by established principles of structural dynamics and soil mechanics. Such an analysis must demonstrate that the exterior utility system will withstand, without disrupting service, the ground accelerations induced in the system by a major seismic event. The effect of such an event on the system will be determined using either acceleration-time history records or equivalent response spectra of major seismic events such as the May 18, 1940, El Centro earthquake. The actual earthquake record or response spectra used, including artificially generated spectra, will be seismologically appropriate to the site and may be scaled in amplitude for maximum base acceleration as determined by the earthquake history of the area and by the principles of engineering seismology.

12-3. Earthquake considerations for utility systems. *a. Earthquake-Resistant Facilities.* A fundamental precept of seismic design is that it is virtually impossible to design facilities to resist every earthquake. Some damage must always be expected. The proper emphasis for good seismic design of exterior utility systems should then be on the development of earthquake-resistant facilities for which measures have been taken to limit damage and to provide for expedient restoration of service. The two most important parameters in evaluating the seismic resistance of utility systems are site geology and structural configuration.

b. Site Geology. The geology beneath a facility exerts considerable influence on the magnitude of the surface accelerations experienced during an earthquake. Current seismic building codes generally recognize this by taking soil type into account in seismic design (e.g., S factor in chap 3). The best material on which to construct a utility system, from a strictly seismic standpoint, is sound rock. Unconsolidated sand or soft clay present the greatest hazards. Unconsolidated materials, either native soil or fill, present hazards of uncontrolled or differential settlements. Even when utilities are built on good soils, considerable structural difficulties can develop. The interface between native soil and engineered fill can present serious earthquake hazards if the fill is improperly compacted or is improperly benched or terraced. Seismically induced relative movement of the fill with respect to the native material can, through settlement or through slippage at the fill-native material interface, shear off an underground utility pipe.

c. Structural Configuration. Structurally flexible underground systems have better earthquake resistance than rigid systems. Underground utilities can often be displaced during an earthquake, despite the relatively large-magnitude forces that may be required to initiate movement. A flexible system, designed to permit some relative movement, will be less apt to fail during a major earthquake. Utility pipes, rigidly attached to appurtenances, can be sheared off by seismically induced differential settlements between the appurtenance structure and the adjoining pipes. Flexibility should be provided in utility pipes at entrances and exits to heavy, rigid appurtenances, and especially in systems dependent upon sound, uncracked pipe and connections for satisfactory performance. The same is true for pipes passing from native material into engineered fill. While it is not feasible to design the utility pipe to support some portion of the fill, the pipe can be made flexible at the interface to thus accommodate some relative movement.

12-4. General planning considerations. The considerations presented herein are guidelines for the planning of earthquake-resistant facilities. Since some damage should always be expected with major seismic activity, the considerations given here stress procedures to be followed to lessen the effects of seismic activity on utility systems and service.

a. Municipal-size facilities should be planned and designed with due regard for possible seismic emergencies; disaster plans and equipment which may be required should be anticipated. Examples of emergency provisions and policies which may be anticipated in the planning stage are as follows:

(1) Specialized emergency equipment, such as mobile flame ionization detectors necessary for the detection of gas leaks, should be available.

(2) Structures that may be used as emergency operation centers should be equipped with battery or other standby power supply systems for communication with emergency vehicles by two-way radio.

(3) Provision should be made for the procurement of gasoline for emergency vehicles. Manually operated fuel pumps should be provided for use in pumping gasoline in the event of power failure.

(4) Emergency battery and/or gasoline driven generator-powered lights should be provided for use in restoring utility service in the event of a power failure.

(5) The engineering staff responsible for the utility system should, from time to time, bring the emergency seismic disaster plans up to date.

(6) Seismic disaster plans should include contingency plans defining procedures for dealing with fires, landslides, and possible health hazards resulting from disrupted sanitary facilities.

b. Individual Facilities. Examples of earthquake disaster procedures that may be implemented into the design in the planning stage are as follows:

(1) Persons having responsibility for the supervision and maintenance of critical facilities should establish earthquake disaster plans. Such plans will be subject to the approval of the utility authority.

(2) The utility authority should emphasize the importance of seismic disaster plans to the supervisory personnel of critical facilities such as hospitals. Seismic disaster plans should be emphasized to the same extent as fire protection plans.

(3) Capability should be established in critical facilities for water to be supplied from emergency reservoirs or wells.

(4) Personnel should be organized to shut off gas service when necessary and instructed not to restore service until advised to do so by the utility authority. For essential facilities in seismic zones 3 and 4, an approved earthquake actuated gas shut off valve should be provided.

(5) Plans showing the locations of utility service lines in buildings should be kept available for emergencies.

12-5. Specific planning considerations. The requirements given here are intended to be used in the planning of a utility system of either a major facility of municipal size or an individual facility of high priority in seismic areas. These requirements supplement applicable agency manuals.

a. General. Whenever practical, utility piping should avoid unstable ground or known earthquake faults, should not traverse native soil structures having widely varying degrees of consolidation, and should not pass from natural ground to unstable fill.

b. Water. Where possible it is preferable to have at least two independent sources of water supply for municipal-size facilities in Zones 2, 3, and 4 (refer to chap 3, para 3-4 for seismic zone maps). When water is furnished by a public utility company, a secondary supply may be provided from onsite wells or from an onsite reservoir. When the water source consists of an onsite well, an additional well should be drilled at a point as widely separated as is practical from the first well. Decentralization of municipal-size waterworks will provide a more flexible water supply network and thus promote a more dependable water supply during a disruptive earthquake. Where practicable, onsite water distribution systems in Zones 2, 3, and 4 should be laid out in a grid pattern. In the event service is disrupted in one section of the grid, water may be drawn from any of several adjacent sections. The grid will be valved to permit the isolation of breaks and to facilitate the emergency distribution of water (e.g., fig 12-8).

c. Gas. Provisions will be made such that installations normally supplied by public utility systems in Zones 2, 3, or 4 for which a gas outage would be critical can be supplied by a liquid petroleum gas (LPG) standby system. Gas distribution networks in Zones 1, 2, 3, and 4 will be valved so that breaks in gas lines may be isolated.

d. Power. Two independent sources of support are less likely to be available for electrical distribution systems than for water and gas supply systems. For Zones 2, 3, and 4, standby power generating facilities should be maintained for use in critical areas such as essential systems for hospitals, computer centers, communication systems, etc., in the event of normal power supply disruption. Such standby systems may consist of diesel or gasoline engine driven electric generators located within the building.

e. Sanitary Sewers. The design of sewer systems for municipal-size facilities located in Zones 2, 3, and 4 will incorporate provisions to eliminate as much as practicable the possibilities of wastewater flooding, contamination of groundwater, and contamination of open water storage reservoirs, should rupture occur to sewers and sewage disposal structures. The design of sewage treatment facilities in Zones 2, 3, and 4 will consider the possibility of decentralizing treatment facilities to minimize possible damage. The practicability of decentralization will be weighed against increased operating, maintenance, and initial costs. In Zones 2, 3, and 4 a means will be provided to rapidly empty and bypass sewage treatment and sewage pumping plant facilities. Should it be impossible to dump raw sewage into emergency outfalls, some simple method of treating the raw sewage should be provided to safeguard health and prevent a nuisance. Mobile pumping equipment should be available for pumping raw sewage into the nearest sewer collector in the event of a pumping plant breakdown.

f. Storm Sewers. More damage to storm sewers and storm sewer facilities can be tolerated than for sanitary sewers and sewage disposal facilities. Cracked or damaged storm sewers in most instances present little danger to health or property. In certain areas where damage to equipment can result from flooding or from infiltration and settlement of fill, care in the design of the storm sewer system must be taken in order to minimize the possibility of cracked or broken pipes.

g. Miscellaneous Systems. It is not feasible to provide secondary distribution systems for central steam, motor vehicle fuel, air, and similar utility systems, but all planning considerations given above, where applicable, will apply to these systems.

12-6. Design considerations. The provisions of this paragraph are intended to supplement rather than supersede the provisions of the various military design manuals and other applicable government criteria.

a. Materials and Construction. Specifications for materials and construction will be governed by the applicable government criteria.

b. Pipe Flexibility. No section of a pipe in Zones 2, 3, or 4 will be held fixed while an adjoining section is free to move, without provisions being made to relieve strains resulting from differential movement, unless approved calculations show that the pipeline can resist the stresses caused by the predicted or estimated pipe movements. Flexibility will be provided by the use of flexible joints or couplings (e.g., fig 12-1 through 12-7) at the following points:

(1) Immediately adjacent to both sides of the surface separating different types of soil having widely differing degrees of consolidation.

(2) At all points that can be considered to act as anchors.

(3) At all points of abrupt change in direction, and at all tees.

c. Water. Buildings housing critical functions, such as hospitals, will be provided with two or more service lines. The service lines will be connected to separate sections of the grid so as to provide continued service in the event one section of the grid is isolated. Services will be interconnected in the building with check valves to prevent backflow. Flexible couplings or flexible connections will be used between valves and lines for valve installations on pipes 3 inches or larger in diameter. In remote areas, auxiliary storage would be an acceptable alternative.

d. Gas. When secondary or standby gas supply systems cannot be justified for a site, gas distribution networks for buildings in Zones 2, 3, or 4 housing critical functions dependent upon gas will include an aboveground valved and capped stub. Provision will be made for attachment of a portable, commercial-sized gas cylinder system to this stub. For essential facilities in seismic zones 3 and 4, an earthquake-actuated valve will be provided. Provisions will be made for the expedient restoration of service and for the prevention of pilot light leaks when service is restored. If an earthquake- actuated shutoff valve presents the possibility of disrupted service in buildings where the fire hazard is small, a manually operated shutoff valve will be installed. The location and operation of such a valve will be made known to the supervisory personnel of the building.

e. Power. Individual aboveground components of electrical utility systems will be designed for seismic forces under the provisions of chapter 10. Slack will be provided in underground cables whenever such cables enter or exit rigid appurtenances. The provisions of paragraph 12-6*b* will not be held applicable to underground electrical utility conduits.

f. Storm Sewer Facilities. While it is desirable to have flexibility in storm sewer pipe, such flexibility cannot, in most instances, be provided without inordinate cost. The provisions of paragraph 12-6*b* will not be held applicable to storm sewer pipes.

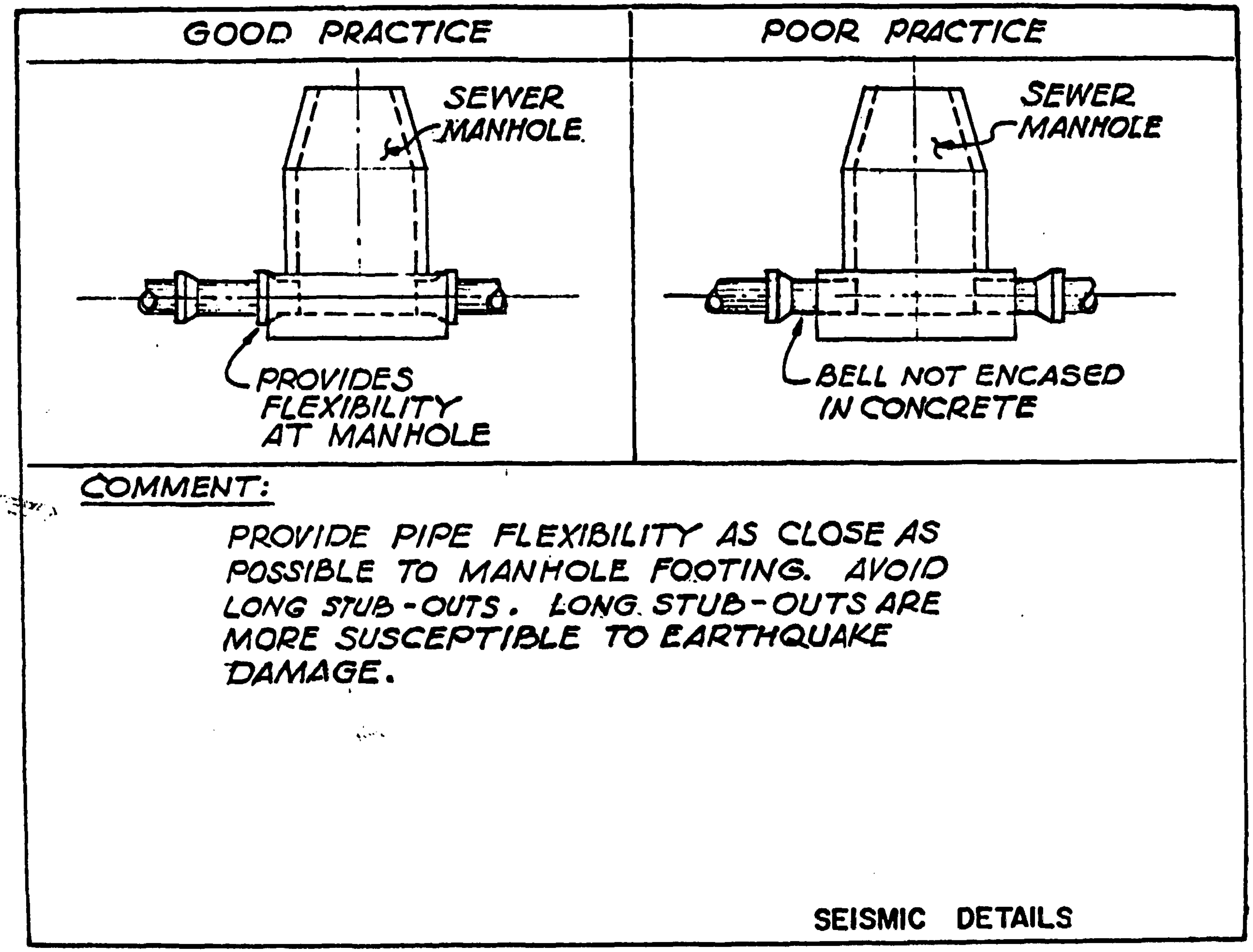
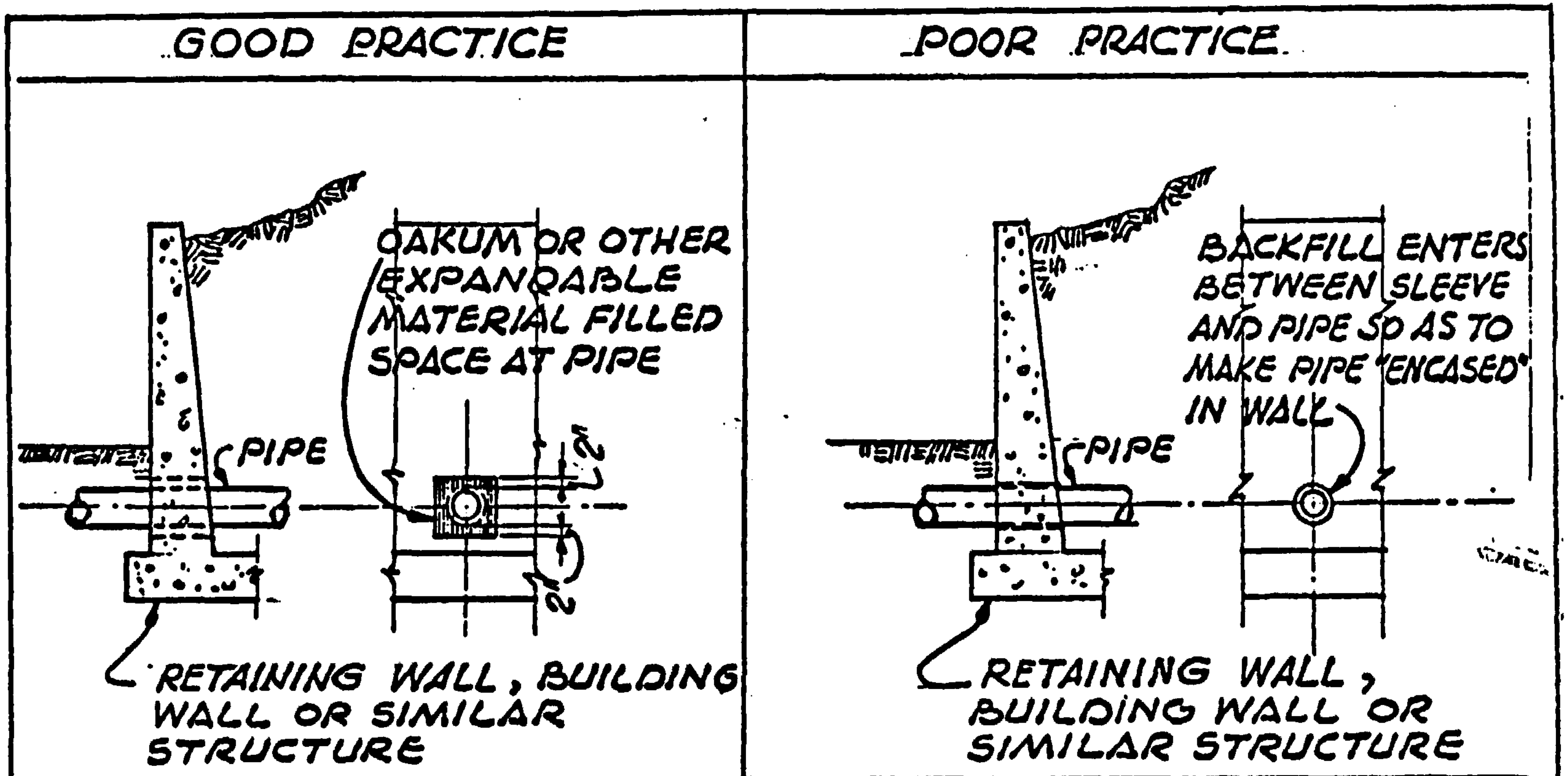


Figure 12-1. Seismic Details



COMMENT:

ALLOW THE PIPE TO PASS THRU WALL WITHOUT RESTRAINT. ANTICIPATE POSSIBLE SETTLEMENT OF WALL BY PROVIDING SUFFICIENT CLEARANCE AROUND PIPE.

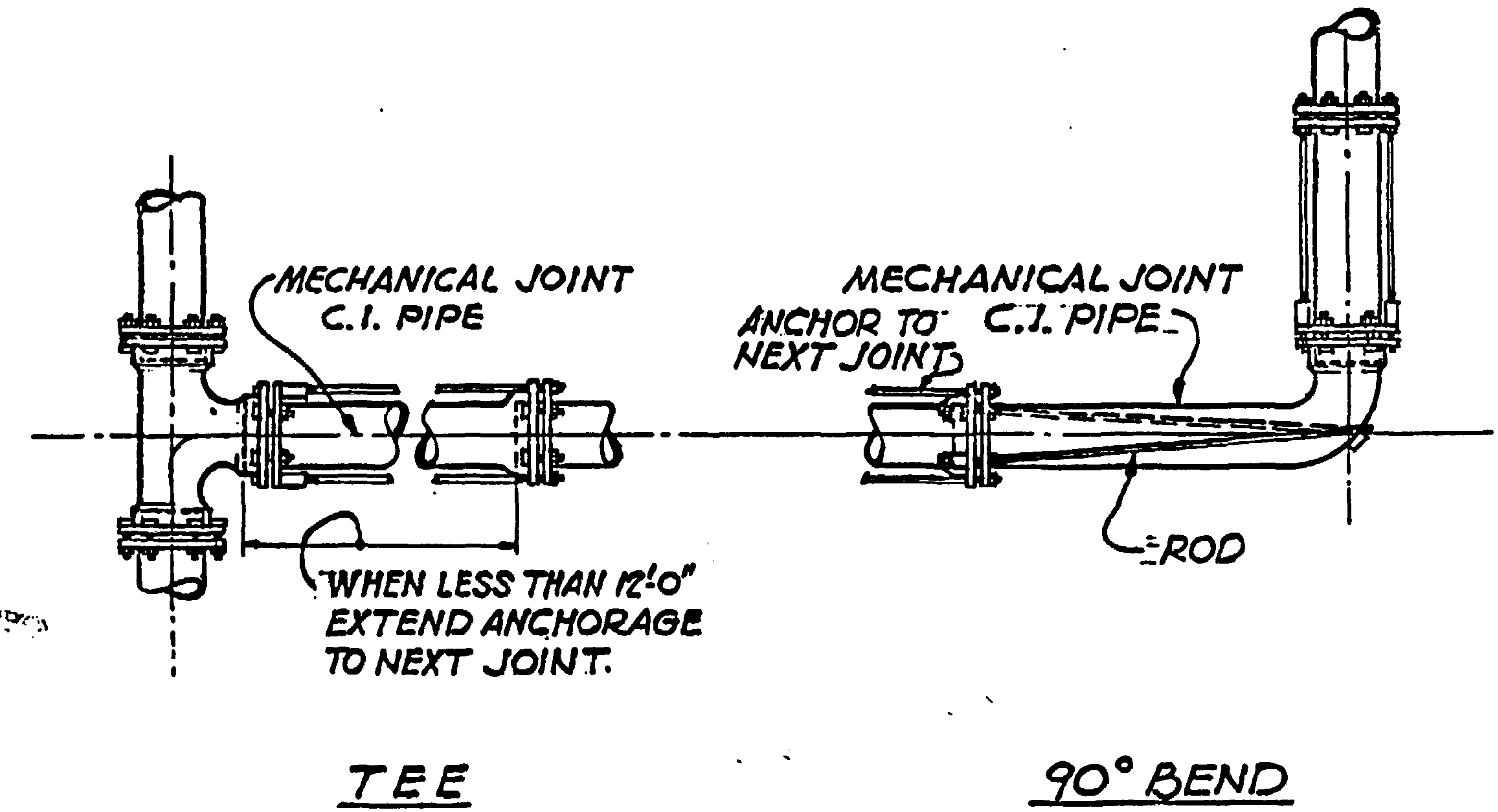
SEISMIC DETAILS

Figure 12-2. Seismic Details

Every attempt, however, should be made to provide flexibility in the connection of storm sewer pipes to rigid appurtenances in Zones 2, 3, and 4.

12-7. As-built drawings. Complete as-built drawings will be required under all contracts for new work for water and gas line installations. Such drawings will show the location of valves and pipelines referenced to permanent structures and existing survey monuments.

12-8. Seismic details. Figures 12-1 through 12-8 are provided to show acceptable seismic details. Some of the plates show examples of good and poor seismic details. Other plates merely illustrate details that have exhibited good seismic details and resistance. Where required by the provisions of this chapter, these recommended seismic details or similar equivalent details will be incorporated in the utility design.



COMMENT:

SHOWN ABOVE ARE TWO TYPES OF ACCEPTABLE FLEXIBLE JOINTS. SINCE ANCHOR BLOCKS ARE NOT REQ'D, FLEXIBLE CONNECTIONS ARE NOT NECESSARY FOR ALL ENDS OF THE TEE.

SEISMIC DETAILS

Figure 12-3. Seismic Details

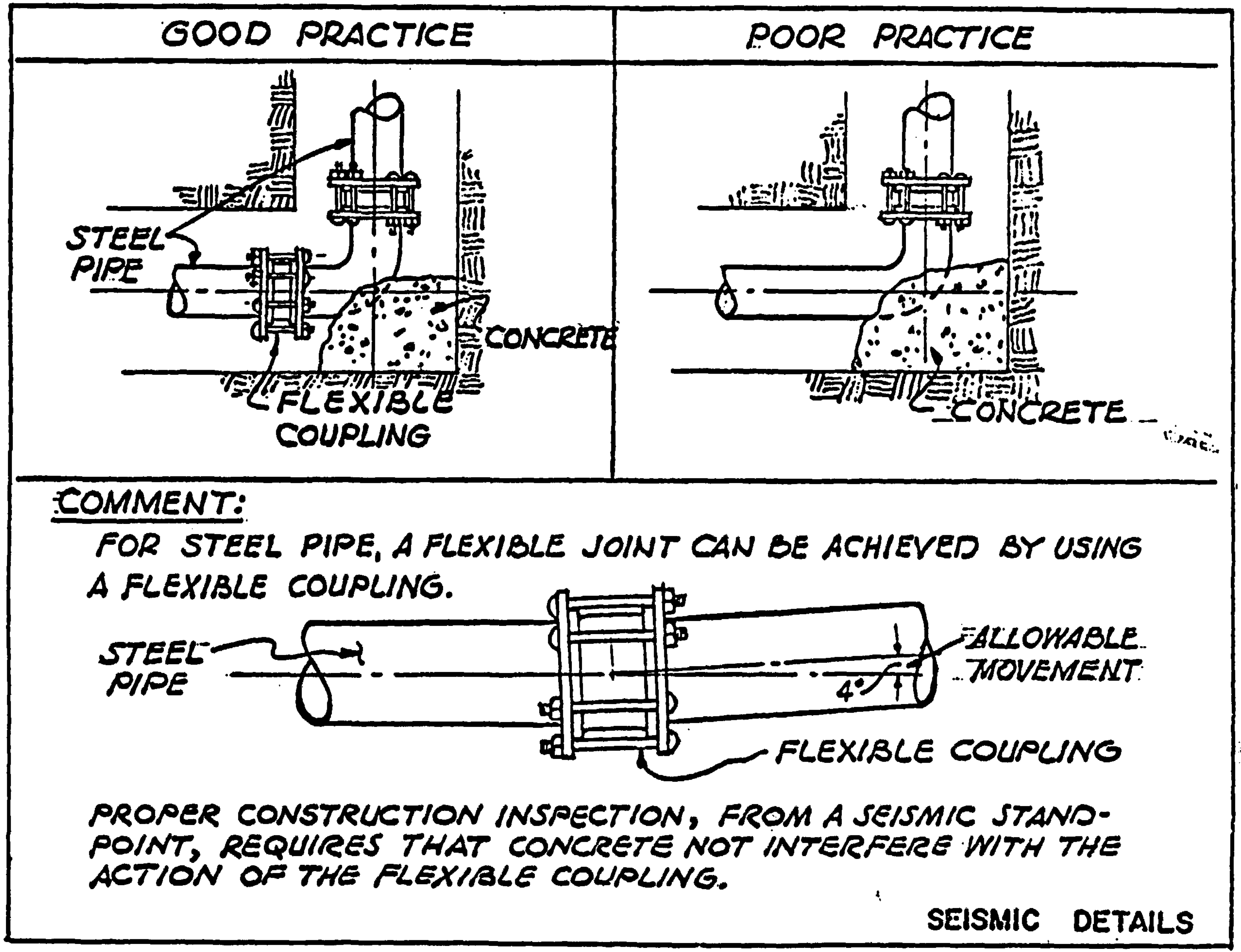
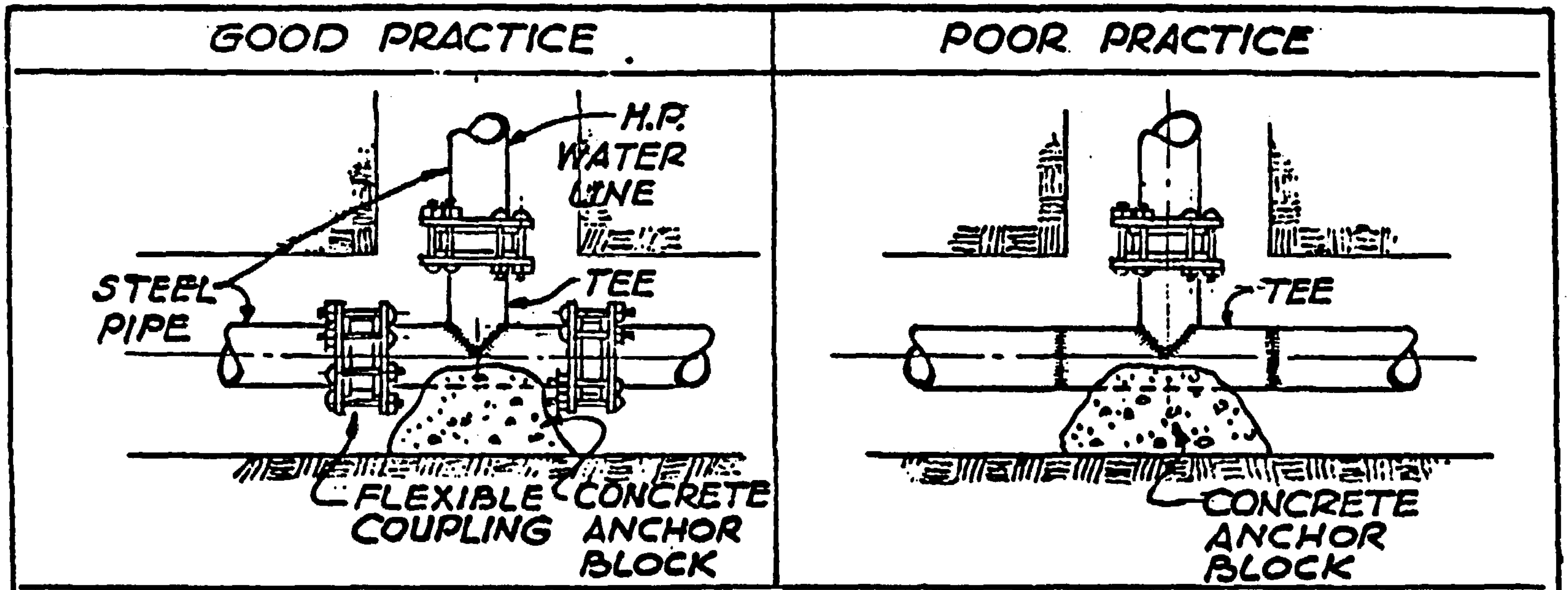


Figure 12-4. Seismic Details



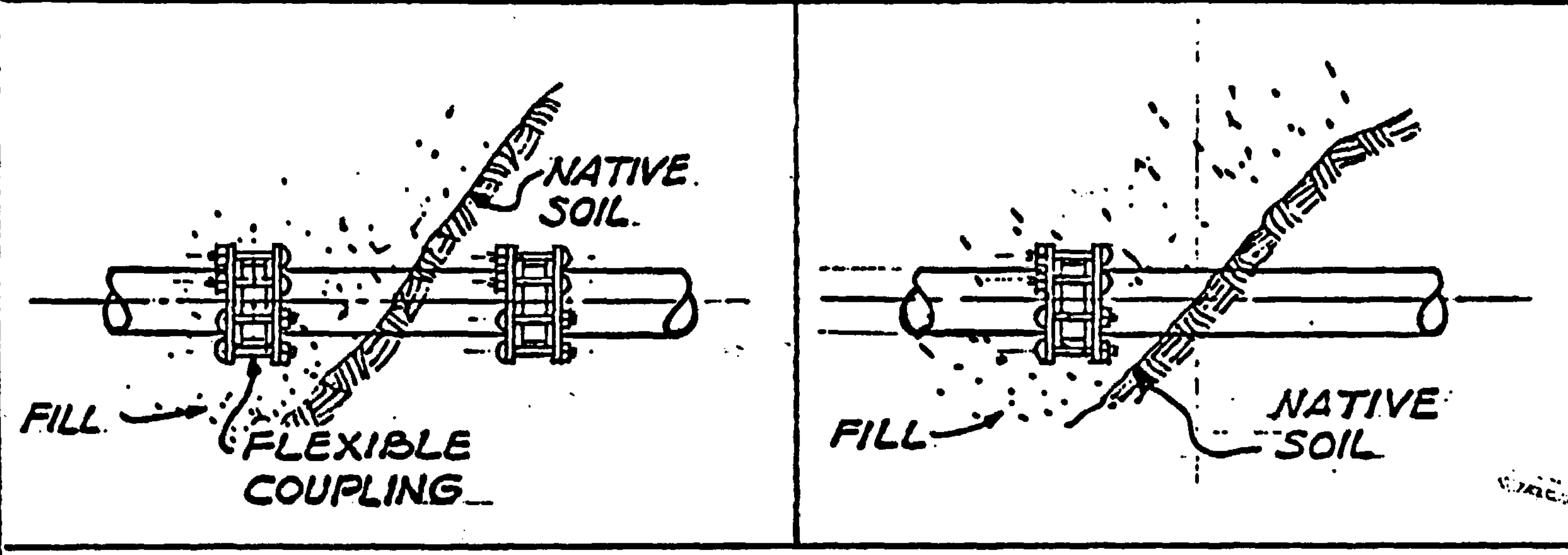
COMMENT:

GOOD SEISMIC DESIGN PRACTICE REQUIRES THE USE OF THREE FLEXIBLE COUPLINGS AT AN ANCHORED TEE. THE CONCRETE ANCHOR BLOCK USED TO PREVENT THE HIGH-PRESSURE WATER LINE FROM SEPARATING ALSO PREVENTS MOVEMENT UNLESS FLEXIBILITY IS PROVIDE BY FLEXIBILE COUPLINGS.

SEISMIC DETAILS

Figure 12-5. Seismic Details

GOOD PRACTICE	POOR PRACTICE



COMMENT:

BETTER FLEXIBILITY IS PROVIDED BY THE USE OF TWO FLEXIBLE COUPLINGS, ONE ON EACH SIDE OF THE SURFACE SEPARATING THE FILL AND NATIVE SOIL.

SEE PLATE 10-4-7 FOR SOME ADDITIONAL TYPICAL FLEXIBLE COUPLINGS.

SEISMIC DETAILS

Figure 12-6. Seismic Details

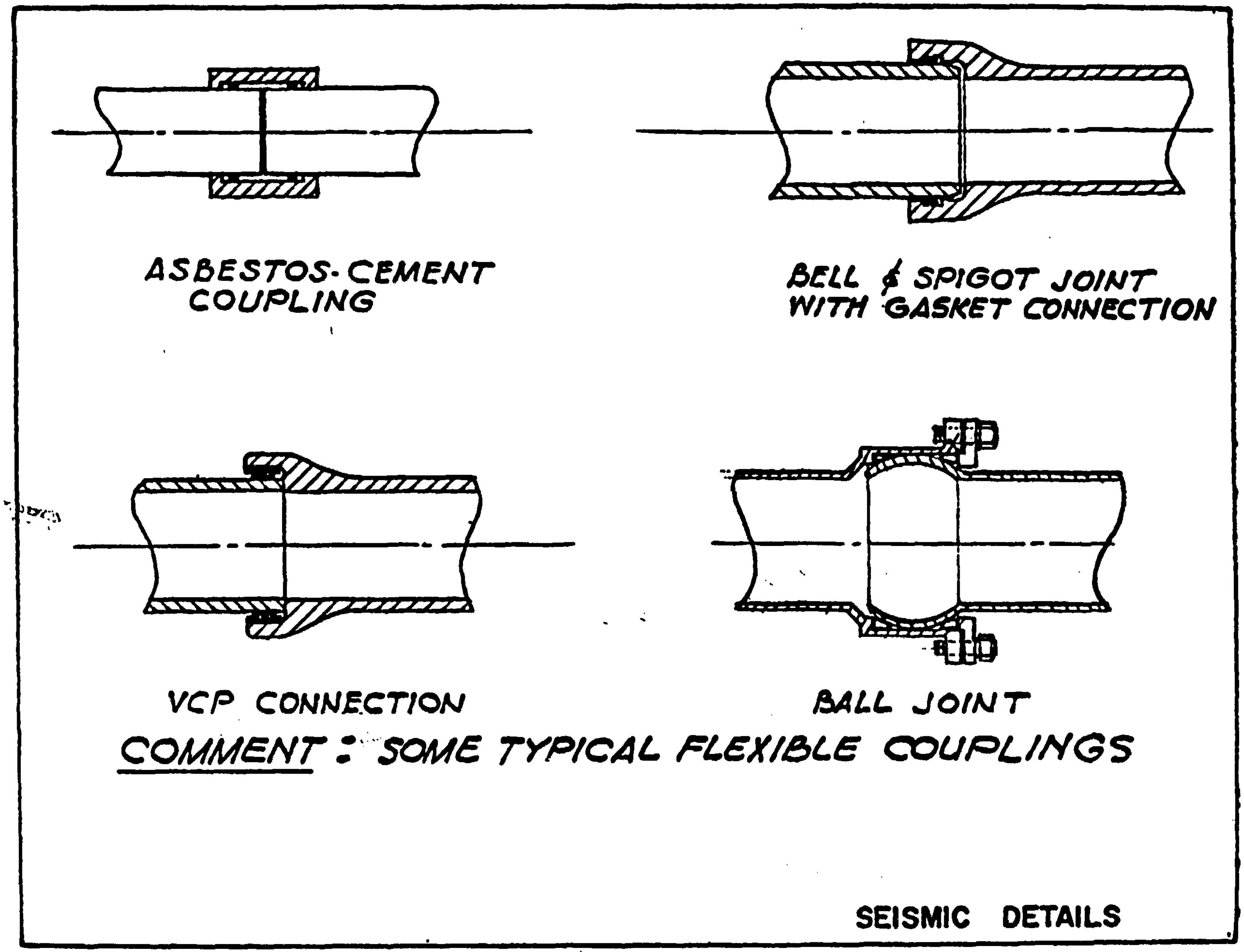


Figure 12-7. Seismic Details

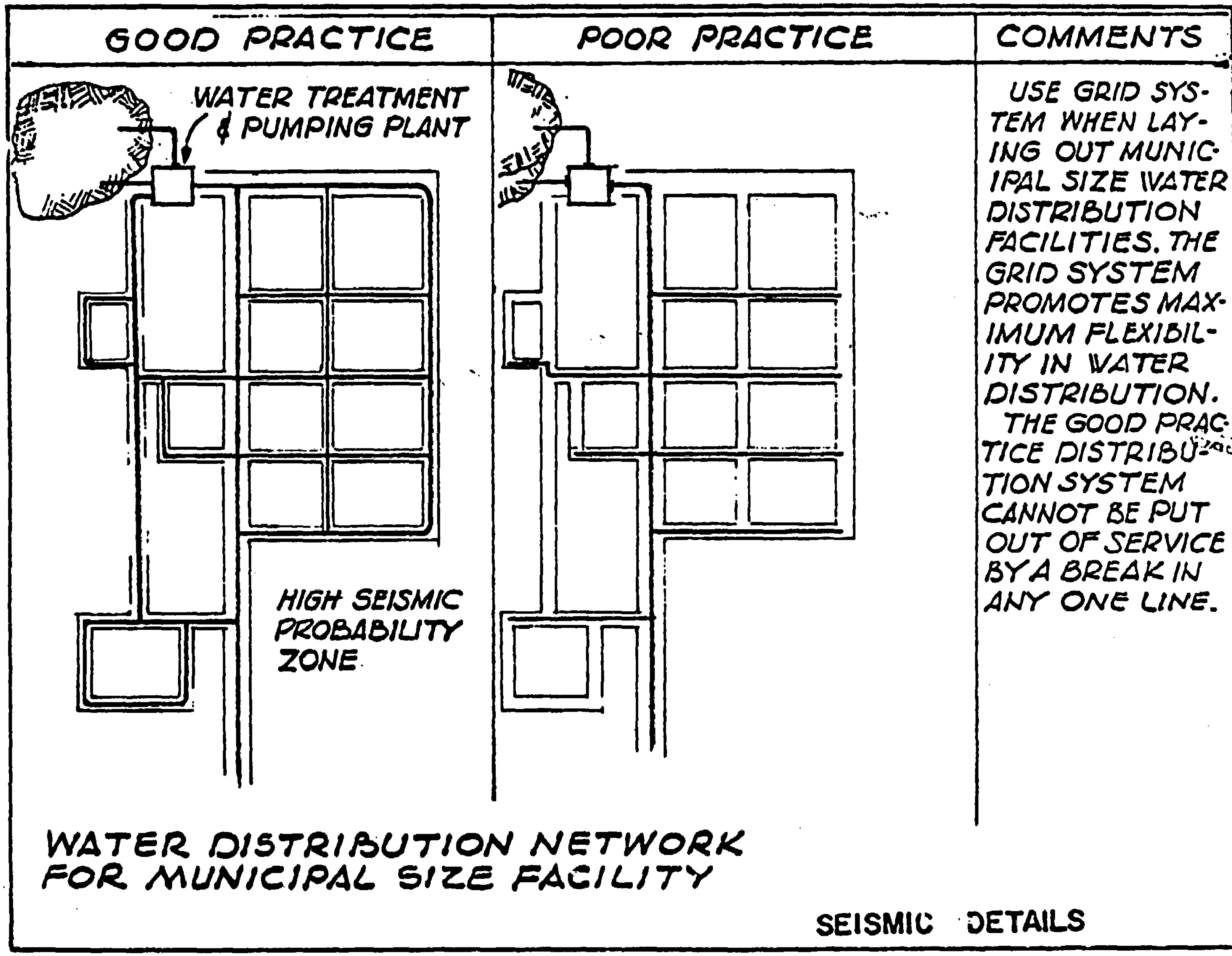


Figure 12-8. Seismic Details

APPENDIX A
STRUCTURAL SYSTEMS

A-1. Purpose and scope. This appendix gives illustrative examples for designing various types of lateral systems. Generally, the calculations determine earthquake lateral forces and their distribution to the resisting elements of the buildings. Some examples are essentially complete, covering frames, walls, diaphragms, and foundations. Examples that are not complete include references to other appendices for examples of shear walls, frames, and diaphragms. Calculations are not given where ordinarily accepted design procedures are involved, such as sizing and detailing members once forces are determined.

A-2. Use of appendixes. The appendixes are purely advisory; they are not intended to place super-restrictions on the manual. The appendixes are not a handbook for the inexperienced designer. Neither the manual, nor the manual supplemented by the appendixes, can replace good engineering judgment in specific situations. Designers are urged to study the entire manual.

A-3. Commentary. *a.* Unless otherwise indicated, all design examples in this appendix are based on Zone 4, where Z = 1.00. But the principles and methods for determining lateral forces are alike for all zones. For instance, lateral forces can be converted for use in other zones simply by multiplying by the value of "Z" required for the applicable zone (viz. 3/4 for Zone 3, 3/8 for Zone 2, and 3/16 for Zone 1).

b. Examples A-1, A-2, A-3, and A-5 are for the same basic building, using (1) bearing walls, (2) concrete frames, (3) steel frames, and (4) frames in combination with shear walls (a dual bracing system) respectively. These examples tend to illustrate the relationship between architectural features (fenestration and materials of construction) and structural design.

c. A 10-pound-per-square-foot weight is added to the roof for the seismic effect of the upper half of the top-story partitions.

d. It is assumed that stairs are detailed so as not to transmit shears from floor to floor. Also, removable and special partitions (such as utility room walls) will be made flexible or isolated so as not to affect the distribution of lateral loads or to act as shear walls.

e. Metal-deck roofs are considered to form flexible diaphragms, and roof loads are distributed according to tributary area rather than relative rigidity of walls below.

A-4. Design examples.

Design Example	Description
A-1	*Box System.* A two-story building with bearing walls in concrete using a series of interior, vertical load-carrying columns and girder bents.
A-2	*Concrete Ductile Moment Resisting Space Frame.* A three-story building with a complete ductile moment resisting space frame in concrete without shear walls.
A-3	*Steel Ductile Moment Resisting Space Frame and Steel Braced Frame.* A three- story building with transverse ductile moment resisting frames and longitudinal frames with K-bracing.
A-4	*Dual Bracing System.* A two-story building in concrete with a ductile moment resisting space frame and with shear walls.
A-5	*Dual Bracing System.* A three-story building with a ductile moment resisting space frame in structural steel and with shear walls in concrete.
A-6	*Wood Box System.* A two-story wood framed building, using wood floor and roof decks, and wood stud walls with plywood sheathing.
A-7	*Special Configuration.* A one-story building with concrete bearing walls on three sides and open on one side.
A-8	*L-Shaped Building.* A three-story building with bearing walls in concrete, using a series of interior vertical load-carrying columns and girder bents.

DESIGN EXAMPLE: A-1

BUILDING WITH A BOX SYSTEM:

Description of Structure. A two-story administration building with bearing walls in concrete, using a series of interior, vertical load-carrying column and girder bents. The structural concept is illustrated on Sheets 3 and 4.

Construction Outline.

Roof:
- Built-up, 5-ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd Floor:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Bearing walls in concrete, furred with GWB finish

Partitions:
- Non-structural removable dry-wall, except concrete as structurally required.

Design Concept. Since the structure is without a complete load-carrying space frame, the K-factor is 1.33. The metal deck roof system forms a flexible diaphragm, therefore the roof loads are distributed to the shear walls by tributary area rather than by second story wall stiffnesses. The roof diaphragm being flexible will not transmit accidental torsion to the shear walls. The metal deck with concrete fill system for the second floor forms a rigid diaphragm. The shear walls react to the forces from the diaphragm, therefore the relative rigidities of the various walls and the individual piers must be determined. This is necessary so that a logical and consistent distribution of story shears to each wall and pier can be made. The wall analysis utilizes the Design Curve for Masonry and Concrete Shear Walls on Figure 6-11.

Discussion. A 10 psf partition load is included in the seismic roof loading but is not included in the vertical design. The stairs are isolated so that they will not transmit shears from floor to floor. The walls along Lines A C 3 & 5 act as vertical cantilever beams joined by struts at the floor lines. The overturning moments are distributed to the individual piers in proportion to the pier stiffnesses. The end wall along Line 7 abuts an existing building, therefore a wall with no openings is provided. The spandrels in wall along Line 1 must be designed to transfer vertical shears due to shear wall action.

Loads.

Roof:

5-ply roofing	=	6.0 p.s.f.
1" insulation	=	1.5
Steel deck	=	2.3
Steel purlins	=	3.7
Steel girders & columns	=	1.2
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load		25.7 p.s.f.

Add for seismic:

Partitions		10.0 p.s.f.
Total for seismic		35.7 p.s.f.*
Live Load		20 p.s.f. (no snow)

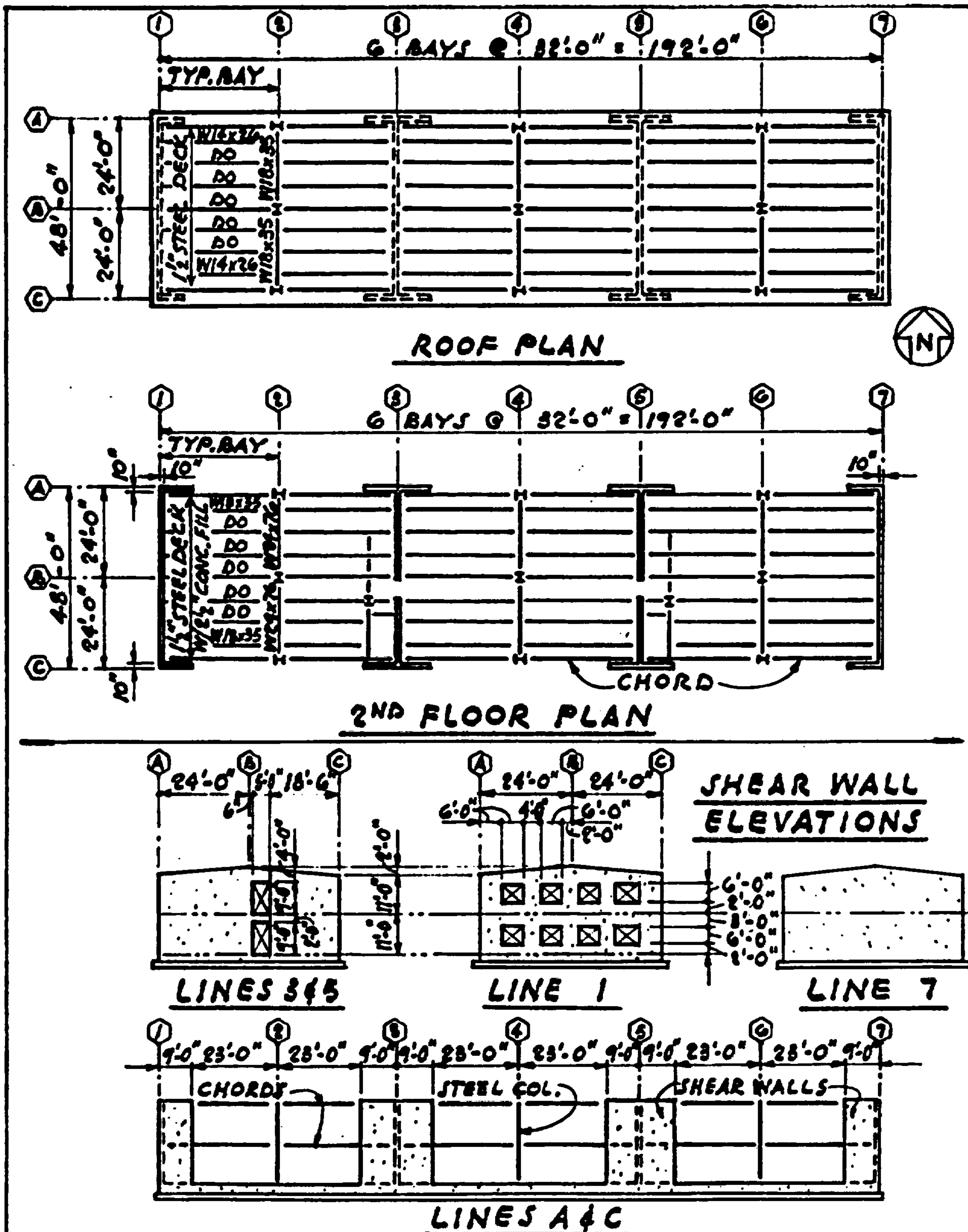
2nd Floor:

Finish	=	1.0 p.s.f.
Steel deck	=	3.1
Concrete fill	=	32.0
Steel beams	=	5.9
Steel girders & columns	=	1.5
Partition	=	20.0
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load	=	74.5 p.s.f.*
Live Load	=	50.0 p.s.f.

Materials.

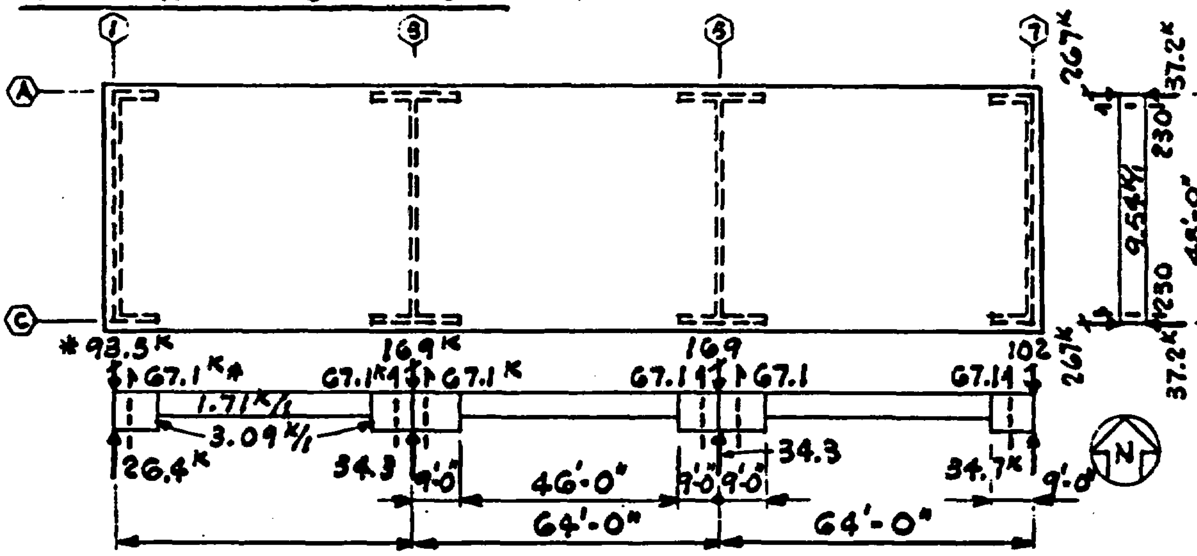
Structural steel .... F_y = 36 k.s.i.
Concrete .... f'_c = 4,000 p.s.i., $E_c = 3.6 \times 10^6$ psi
Reinforcing steel .... f_y = 40,000 p.s.i.
Allowable soil pressure ... = 3,000 p.s.f. Vertical Load
Allowable soil pressure ... = 4,000 p.s.f. Vertical plus Seismic

*Weight of shear walls are not included here. The weight of the concrete shear walls are calculated on pages 4 and 5. The weights of the exterior windows and architectural wall panels are included in the partition weights.



Example A-1 — 3 of 44 — Box System

DISTRIBUTION OF BUILDING WEIGHT TO ROOF DIAPHRAGM @ 100% G



10" CONC. WALLS 0.125 KSF

SIDEWALLS A & C: 5.5' x 0.125 = 0.688 K/l, x 2 = 1.38 K/l

CROSSWALLS 1,3,5 & 7: 6.0' x 0.125 = 0.75 K/l, x (2 x .91 + .76 + 1.0) = 2.69 K/l

WALL ON 1 : .76 x .075 x 46.33' = 26.4 (76% SOLID)

WALL ON 7 : 1.0 x 0.75 x 46.33' = 34.7 (100% SOLID)

WALL ON 3 & 5: .91 x 0.75 x 46.33' = 31.6 (91% SOLID)

WALLS ON A & C:

9' WALLS	0.688 x 9' = 6.2K	x 2 = 12.4K	
18' WALLS	= 12.4K	x 2 = 24.8K	
		37.2K	

	(P.2)	E-W LOADS	N-S LOADS
ROOF	0.0357 KSF	x 192' = 6.85 K/l	x 48' = 1.71 K/l
WALLS		2.69	1.38
		9.54 K/l	3.09 K/l

*SAMPLE CALCULATION OF WALL [1]

DISTRIBUTION OF DIAPH. WT TO WALL [1] = 1.71 K/l x 64'/2 = 54.7 } 67.1K

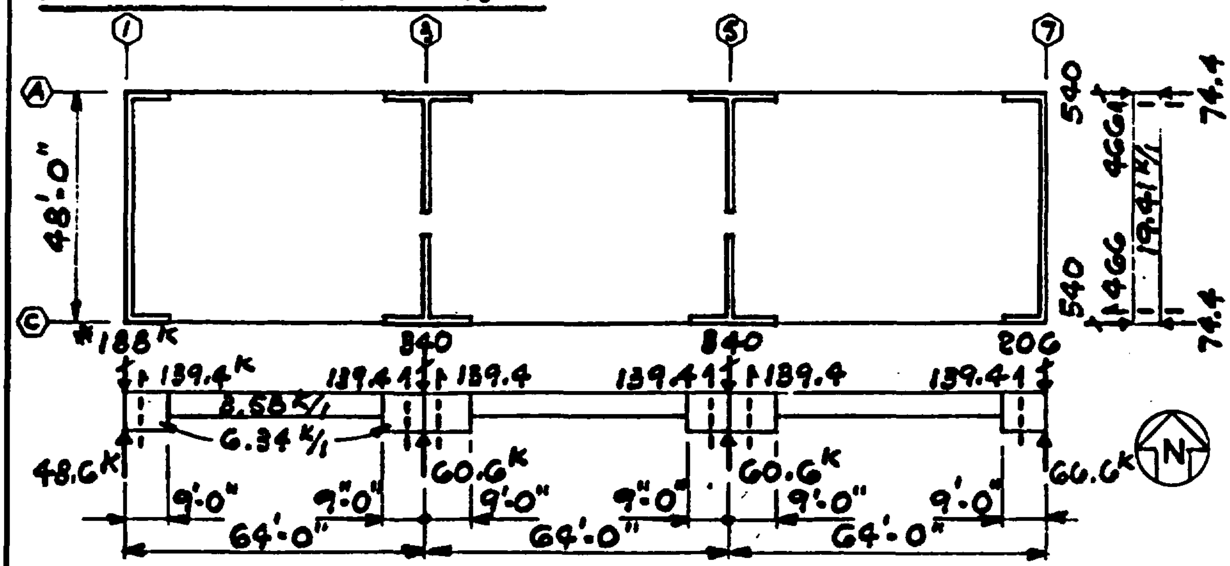
" " SIDEWALL " " " = 1.38 x 9' = 12.4

THE WT CONTRIBUTION FROM IN-PLANE SHEAR WALL [1] (SEE ABOVE) 26.4

THE TOTAL WT CONTRIBUTION TO SHEAR WALL [1] 93.5K

TOTAL TRIB. WT OF ROOF & ALL WALLS W_R = 534K @ 100% G

DISTRIBUTION OF BUILDING WEIGHT AT 2ND FLOOR DIAPHRAGM @ 100 % G



10" CONC. WALLS = 0.125 KSF

WALL ON 1 : .73 × 0.125 × 11.5' × 46.33 = 48.6K

WALL ON 7 : 1.0 × 0.125 × 11.5 × 46.33 = 66.6K

WALLS ON 3 & 5 : .91 × 0.125 × 11.5 × 46.33 = 60.6K

WALLS ON A & C:

9' WALLS 0.125 × 11' × 9' = 12.4K × 2 = 24.8K

18' WALLS = 24.8K × 2 = 49.6K

74.4K

	(P.2)	E-W LOADS	N-S LOADS
FLOOR	0.0745 KSF × 192' = 14.30 K/1		× 48' = 3.58 K/1
WALL	0.73 × 0.125 KSF × 11.5' = 1.06		2 × 0.125 × 11' = 2.76
	1.0 × 0.125 KSF × 11.5' = 1.44		6.34 K/1
	2 × .91 × 0.125 KSF × 11.5' = 2.62		
	19.41 K/1		

*SAMPLE CALCULATION FOR WALL [1]

DISTRIBUTION OF DIAPH. WT TO WALL [1] = 3.58 K/1 × 64/2 = 114.6 } = 139.4K

" " SIDEWALL " " " = 2.76 × 9' = 24.8 }

THE WT CONTRIBUTION FROM THE IN-PLANE SHEAR WALL [1] (SEE ABOVE) = 48.6

THE TOTAL WT CONTRIBUTION TO SHEAR WALL [1] = 188.0K

TOTAL TRIB WT OF 2ND FLR DIAPH & 2ND FLR TRIB WALLS W = 1080K @ 100% G

LATERAL FORCES - EAST-WEST DIRECTION

V = ZIKCSW (FORMULA 3-1)
Z = 1.0 (ZONE 4, TABLE 3-1)
I = 1.0 (TABLE 3-2)
K = 1.33 (BOX SYSTEM, TABLE 3-3)
C = $1/15\sqrt{T}$ (FORMULA 3-2)
S = 1.5 (T_S UNKNOWN, ∴ S = MAX. VALUE)
h = 22 FT.

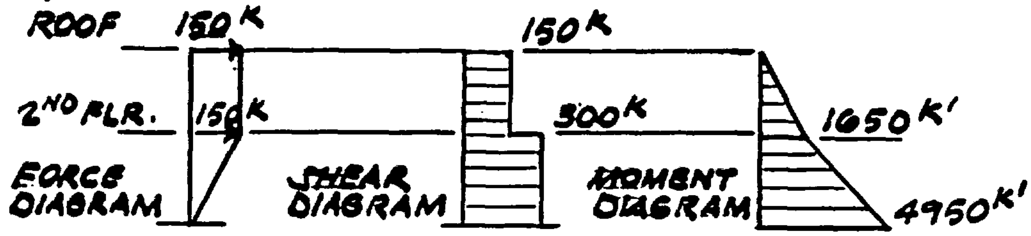
DIRECTION	D	$T = \frac{0.05h}{\sqrt{D}}$	$C = \frac{1}{15\sqrt{T}}$	CS
E - W	192'	.0794	.237	.355

$F_t = 0$ SINCE $T < 0.7$ NEED NOT BE GREATER THAN .14

V = ZIKCSW = 1 × 1.0 × 1.33 × 0.14 × W = 0.186 W

LEVEL	h FT.	Δh FT.	w K	Σw K	wh K-FT.	$\frac{wh}{\Sigma wh}$	F_x K	V K	ΔM_{OT} K-FT.	M_{OT} K-FT.
R	22		534		11,748	.50	150			
		11		534				150	1650	
2	11		1080		11,880	.50	150			1650
		11		1614				300	3300	
GRD	0									4950
Σ			1614		23,628	1.00	300		4950	

$V = 0.186 \times 1614^K = 300^K$



LATERAL FORCES - NORTH SOUTH DIRECTION

$V = ZIKCSW$ (FORMULA 3-1)
$Z = 1.0$ (ZONE 4, TABLE 3-1)
$I = 1.0$ (TABLE 3-2)
$K = 1.33$ (BOX SYSTEM, TABLE 3-3)
$C = 1/15\sqrt{T}$ (FORMULA 3-2)
$S = 1.5$ (T_S UNKNOWN, ∴ S = MAX. VALUE)
$h = 23$ FT.

DIRECTION	D	$T = \frac{0.05h}{\sqrt{D}}$	$C = \frac{1}{15\sqrt{T}}$	CS
N - S	48'	.166	.164	.246

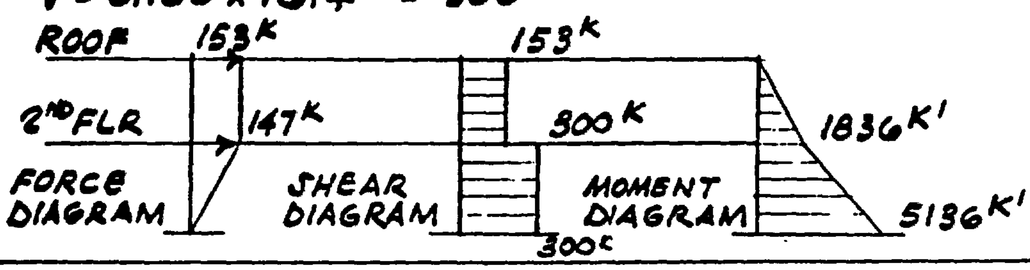
$F_t = 0$ SINCE $T < 0.7$

NEED NOT BE GREATER THAN .14

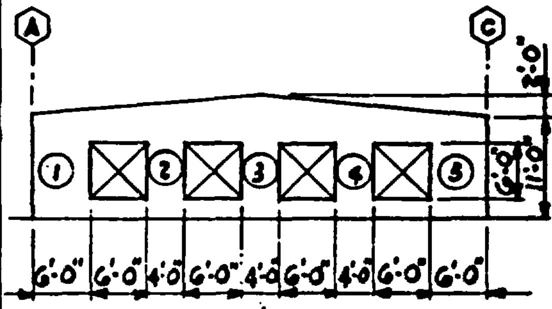
$V = ZIKCSW = 1 \times 1.0 \times 1.33 \times 0.14 \times W = 0.186W$

LEVEL	h FT.	Δh FT.	w K	Σw K	wh K·FT.	$\frac{wh}{\Sigma wh}$	F_x K	V K	ΔM_{OT} K·FT.	M_{OT} K·FT.
R	23		534		12282	.51	153			
		12		534				153	1836	
2	11		1080		11,880	.49	147			1836
		11		1614				300	3300	
GRD	0									5136
Σ			1614		24162	1.00	300		5136	

$V = 0.186 \times 1614^K = 300^K$

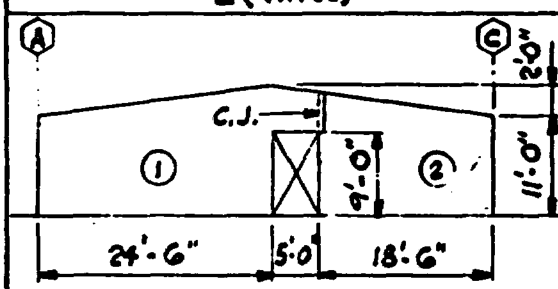


RELATIVE RIGIDITIES (REFER TO EX. C-4)
2ND STORY WALLS



WALL 1 - 2ND STORY

WALLS	PIER	H	D	H/D	Δ FIG. G-11	R	ΣR WALL
WALL 1 - 2ND STORY	1 & 6 (CORNER FIXED)	6'	6	1.0	0.09	11.1 × 2 PIERS = 22.2	
	2,3,4 (RECT. FIXED)	6'	4'	1.5	0.22	4.5 × 3 PIERS = 13.5	
						ΣR=35.7	
					Δ(1-5) = 1/R = 0.028		
	SOLID WALL (CORNER CANT)	12' AVE	48'	0.25	0.018		
$\Delta(WALL) = 0.018 - 0.008 + 0.028 = 0.038$	SUBTRACT BAND @ WINDOW (CORNER CANT)	6'	48'	0.125	(0.008)		
$R(WALL) = \frac{1}{\Delta(WALL)} = 26.3$					ΣΔ(WALL) = 0.038		$\Sigma R = \frac{1}{\Sigma\Delta} = 26.3$



WALL 3 - 2ND STORY
(WALL 5 SIM.)

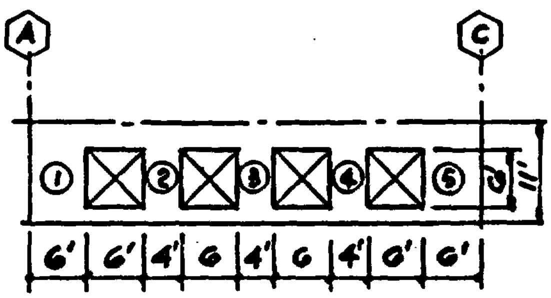
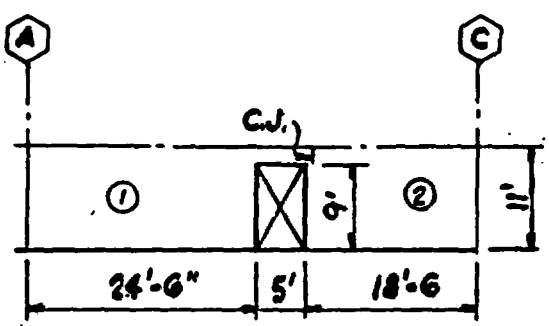
WALLS	PIER	H	D	H/D	Δ FIG. G-11	R	ΣR WALL
WALL 3 - 2ND STORY (WALL 5 SIM.)	1 (CORNER CANT.)	12' AVE	24.5	0.49	0.044	22.7	
	2 (CORNER CANT.)	12'	18.5	0.65	0.065	15.4	
						Σ 38.1	ΣR = 38.1

FOR THIS EXAMPLE, CONTROL JT. IS PROVIDED TO MAKE WALL MORE FLEXIBLE, THEREBY DISTRIBUTING MORE LOAD TO WALL 7

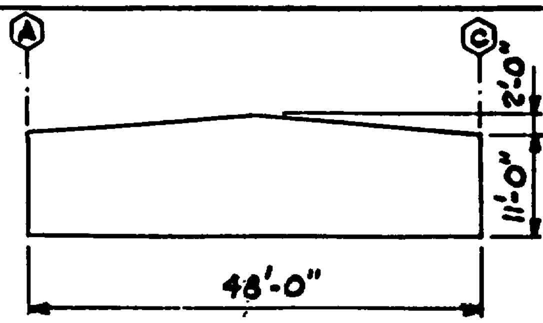
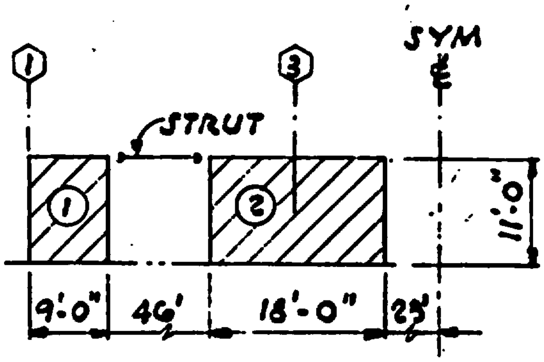
NOTE: SINCE ALL WALLS ARE THE SAME THICKNESS (i.e. 10") THE VALUES FROM FIG. G-11 FOR 12" WALLS MAY BE USED FOR RELATIVE RIGIDITIES WITHOUT ADJUSTMENT.

RELATIVE RIGIDITIES (REFER TO EX. C-4)

1ST. STORY WALLS

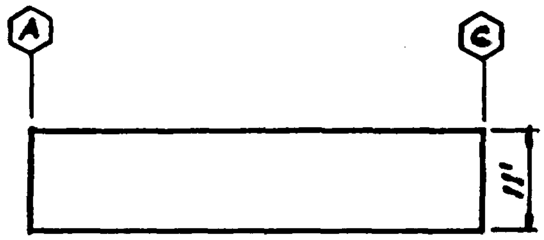
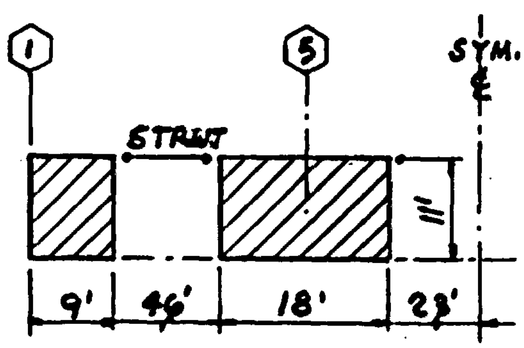
WALL	PIER	H	D	H/D	Δ FIG. C-II	R	ΣR WALL
	1 & 5 (CORNER CANT.)	6	6	1.0	0.09	11.1 × 2 PIERS = 22.2	
WALL 1 1ST STORY	2,3,4	6	4	1.5	0.22	4.5 × 3 PIERS = 13.5; ΣR = 35.7	
					Δ(1-5) = 1/R = 0.028		
	SOLID WALL (CORNER CANT.)	11'	48'	.23	0.017		
Δ(WALL) = 0.017 − 0.008 + 0.028 = 0.037	SUBTRACT BAND @ 6' WINDOW (CORNER CANT.)		48'	0.125	⟨0.008⟩		
$\Sigma R_{WALL} = \frac{1}{0.037} = 27$					ΣΔ (WALL) = .037		$\Sigma R = \frac{1}{\Sigma \Delta W} = 27$
	1 (CORNER CANT)	11'	24.5	0.45	0.037	27.0	
WALL 3 - 1ST STORY (WALL 5 - SIM.)	2 (CORNER CANT.)	11'	18.5	0.59	0.056	17.9; 44.9	ΣR = 44.9

RELATIVE RIGIDITIES (REFER TO EX. C-4)
2 ND STORY

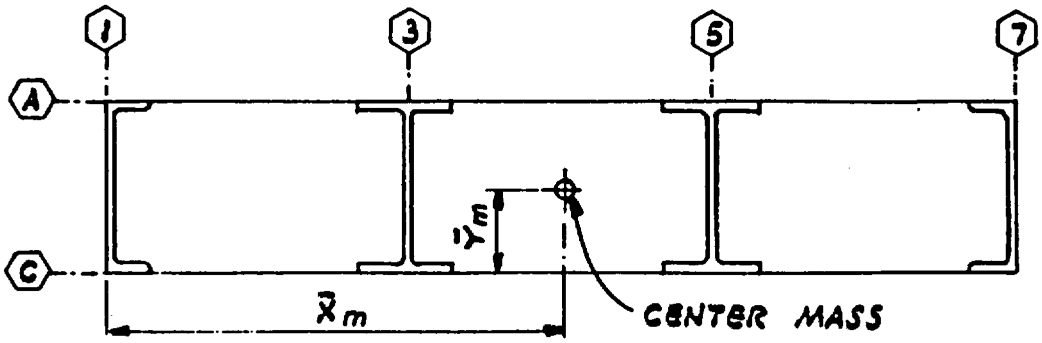
WALL	PIER	H	D	H/D	Δ FIG. 6-11	R	ΣR WALL
 WALL 7 - 2ND STORY	1 CORNER CANT	12 AVE	48	0.25	0.018	55.5	55.5
 WALL C - 2ND STORY (WALL A - SIM.)	1, 4 CORNER CANT	11	9	1.22	0.22	4.5 × 2 PIERS 9.1	
	2, 3 RECT. CANT	11	18	0.61	0.075	13.3 × 2 PIERS 26.6	ΣR: 35.7

RELATIVE RIGIDITIES (REFER TO EX. C-4)

1ST STORY WALLS

WALL	PIER	H	D	H/D	Δ FIG C-11	R	ΣR WALL
 WALL 7 1ST. STORY	1 (CORNER CANT.)	11'	48'	0.23	.017	58.8	58.8
 WALL C - 1ST STORY (WALL A SIM.)	1,4 (CORNER CANT.)	11'	9'	1.22	0.22	4.5× 2 PIERS = 9.0	
	2,3 (RECT. CANT.)	11'	18'	0.61	0.075	13.3× 2 PIERS = 26.6	ΣR = 35.6

CENTER OF MASS AND CENTER OF RIGIDITY
ROOF DIAPHRAGM



			CENTER OF MASS			CENTER OF RIGIDITY		
	X	Y	W (P.4)	W·Xm	W·Ym	RIGIDITY R	R·Xr	R·Yr
WALL 1	0.42'		93.5	39				
WALL 3	64		169	10816				
WALL 5	128		"	21632				
WALL 7	191.58		102	19541				
			533.5	52028				
WALL A		47.58	267		12704			
WALL C		0.42	"		112			
			534		12816			

CENTER OF MASS OF ROOF DIAPHRAGM:

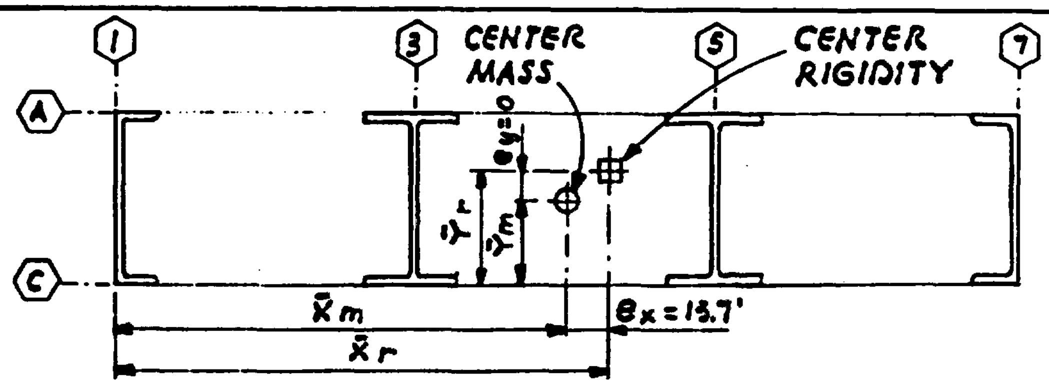
$$\bar{X}_m = \frac{52028}{533.5} = 97.5 \qquad \bar{Y}_m = \frac{12816}{534} = 24'$$

CENTER OF RIGIDITY:

CALCULATIONS NOT REQUIRED SINCE ROOF DIAPHRAGM IS FLEXIBLE AND SEISMIC FORCES ARE DISTRIBUTED BY TRIBUTARY AREA.

CENTER OF MASS OF ROOF DIAPHRAGM IS REQUIRED SINCE THE ECCENTRICITY OF THIS MASS EFFECTS THE TORSIONAL FORCE ON THE RIGID 2ND FLOOR DIAPHRAGM BELOW.

CENTER OF MASS AND CENTER OF RIGIDITY
2ND FLOOR DIAPHRAGM



	X	Y	CENTER OF MASS			CENTER OF RIGIDITY		
			W (P.8)	W·Xm	W·Ym	RIGIDITY R (P.10)	R·Xr	R·Yr
WALL 1	0.42		188	79		27	11	
WALL 3	64		340	21760		44.9	2874	
WALL 5	128		"	43520		"	5747	
WALL 7	191.58		206	39465		58.8	11265	
			*1074	104824		175.6	19897	
WALL A		47.58	540		25693	35.6	NONE	1694
WALL C		0.42	"		227	"	NONE	15
			*1080		25920	71.2		1709

* TOTAL WTS. DO NOT CORRESPOND DUE TO ROUNDING OFF.

CENTER MASS: $\bar{X}_m = \frac{104824}{1074} = 97.6'$ CENTER RIGIDITY: $\bar{X}_r = \frac{19897}{175.6} = 113.3'$

$\bar{Y}_m = \frac{25920}{1080} = 24'$ $\bar{Y}_r = \frac{1709}{71.2} = 24'$

ECCENTRICITY OF ROOF MASS W/RESPECT TO 2ND FLR. CENTER RIGIDITY

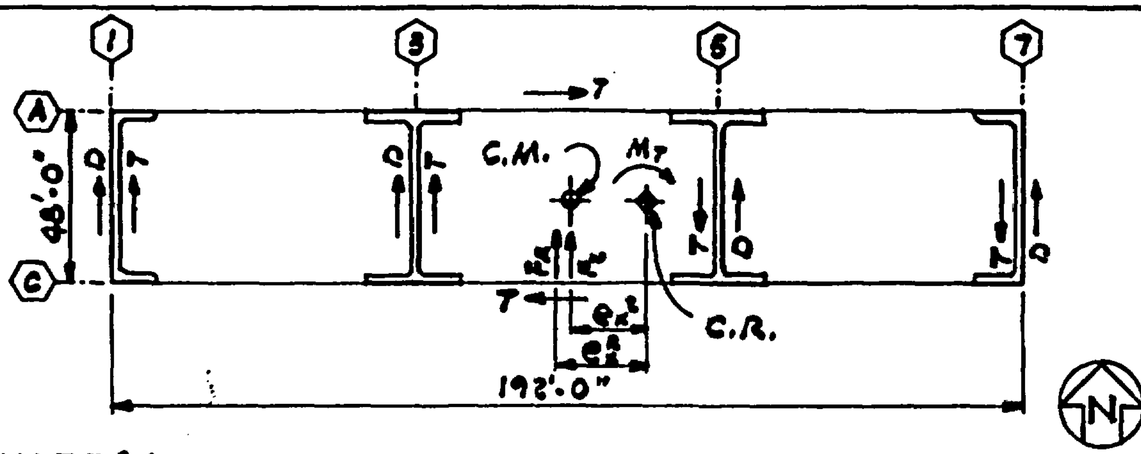
$e_x = 113.3 - 97.5 = 15.8'$ $e_y = 24' - 24' = 0$

(P.12)

ECCENTRICITY OF 2ND FLR MASS W/RESPECT TO 2ND FLR. CENTER RIGIDITY

$e_x = 113.3' - 97.6' = 15.7'$ $e_y = 24' - 24' = 0$

DIAGRAM SHOWING DIRECT SHEAR AND TORSIONAL SHEAR FORCES



NOTES:

C.R. = CENTER OF RIGIDITY
C.M. = CENTER OF MASS
T = TORSIONAL SHEAR FORCE
D = DIRECT SHEAR FORCE
F_R = FORCE FROM ROOF DIAPH (N-S)
F_2 = FORCE FROM 2ND DIAPH (N-S)
e_x^R = ECCENTRICITY OF ROOF MASS W/ RESPECT TO 2ND FLR C.R
e_x^2 = " " 2ND FLR " " " " " " "
M_T = TORSIONAL MOMENT

NORTH-SOUTH DIRECTION

$$M_T = \Sigma (F_x \cdot e_x) = (\underset{(P.7)}{153^K} \times \underset{(P.13)}{15.8'}) + (\underset{(P.7)}{147^K} \times \underset{(P.13)}{16.7'}) = 4725'^K$$

EAST - WEST DIRECTION

$M_T = 0$ SINCE $e_y = 0$

ACCIDENTAL TORSION [SEE PARA. 3-3 (E)]

$M_T = \Sigma F_x \times .05 \times$ MAX. BLDG. DIMENSION
$= 300^K \times .05 \times 192' = 2880'^K$ (GOVERNS IN E-W DIRECTION)

DISTRIBUTION OF SEISMIC FORCES FROM ROOF DIAPHRAGM TO WALLS BELOW

			TORSIONAL FORCE CALCULATIONS					
	WALL	DIRECT FORCE TO WALL	d	R (p.7)	$R \cdot d^2$	M_T	$\frac{Rd^2}{\Sigma Rd^2} \cdot \frac{M_T}{d}$	DIRECT FORCE + TORSION
N-S DIRECTION	1	26.8^K						26.8^K
TOTAL SHEAR	3	48.4						48.4
BELOW ROOF	5	"						"
$F_x = 153^K$	7	29.2						29.2
(p.7)		153^K						
E-W DIRECTION								
TOTAL SHEAR	A	75^K						75
BELOW ROOF $F_x = 150^K$	C	75						75
(p.6)		150^K						

SEISMIC FORCE TO WALL IS DISTRIBUTED BY TRIBUTARY AREA RATHER THAN WALL STIFFNESS. NO TORSION ASSUMED

SAMPLE CALCULATION:

$$\text{DIRECT FORCE (WALL 1)} = \frac{93.5}{584} \times 153^K = 26.8^K$$

DISTRIBUTION OF SEISMIC FORCES FROM 2ND FLOOR DIAPHRAGM TO WALL BELOW

	WALL	DIRECT FORCE TO WALL	TORSIONAL FORCE CALCULATIONS: d	R (P.8)	R·d²	M_T (P.M)	$\frac{Rd^2}{\Sigma Rd^2} \cdot \frac{M_T}{d}$	DIRECT FORCE + TORSION
N-S DIRECTION	1	50ᴷ	95.58'	26.3	240265	4725	13.7ᴷ	63.7ᴷ
TOTAL SHEAR BELOW 2ND FLR DIAPH.	3	72.3	32'	38.1	39014	"	6.7	79
	5	"	"	"	"	"	"	"
	7	105.4	95.58'	55.5	507022	"	29.0	134.4
$\Sigma F_x = 300^K$ (P.7)		300ᴷ		158				
E-W DIRECTION	A	150ᴷ	23.58'	35.6	19794	2880	2.8	162.8
TOTAL SHR BELOW 2ND FLR DIAPH.	C	150	23.58'	35.6	19794	"	2.8	"
$\Sigma F_x = 300^K$ (P.6)		300ᴷ		71.2	864903			

SAMPLE CALCULATION: DIRECT FORCE (WALL 1) $= \frac{R}{\Sigma R} \cdot V = \frac{26.3}{158} \times 300 = 50^K$

NOTES:

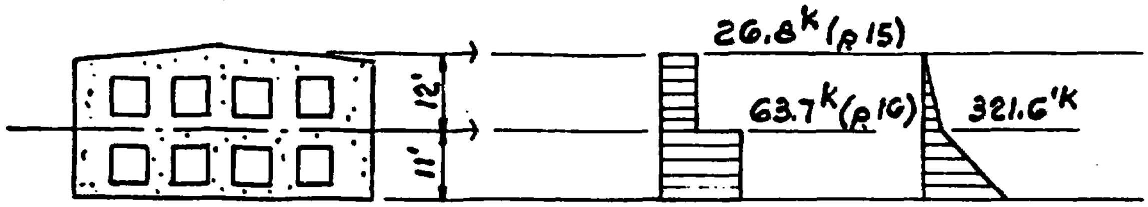
d = DISTANCE OF WALL FROM CENTER RIGIDITY

M_T = THE LARGER OF "CALCULATED" TORSION OR "ACCIDENTAL" TORSION

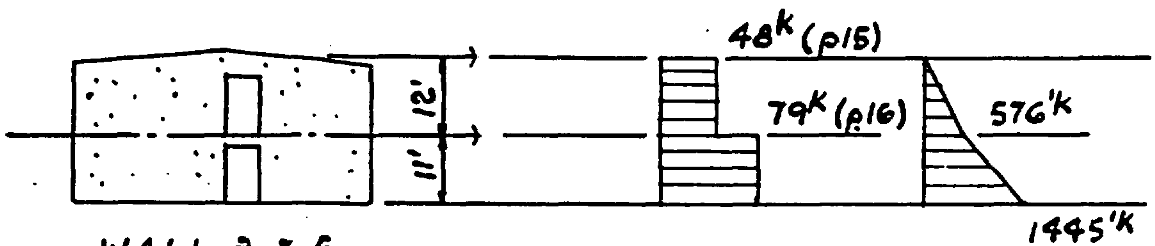
DISTRIBUTION OF SEISMIC FORCES & OVERTURNING MOMENTS

NORTH - SOUTH DIRECTION

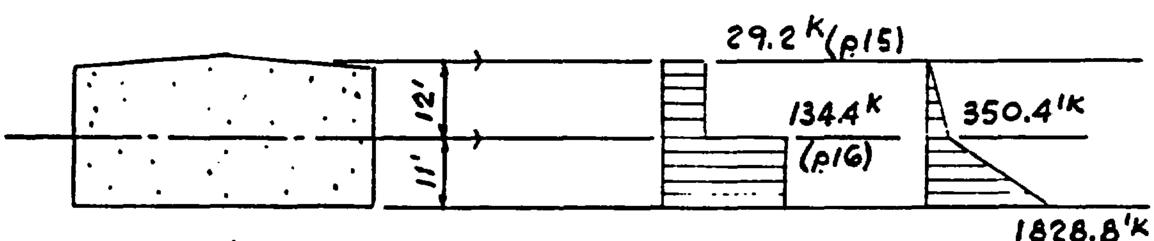
WALL	SHEAR V	OVERTURNING MOMENT MOT



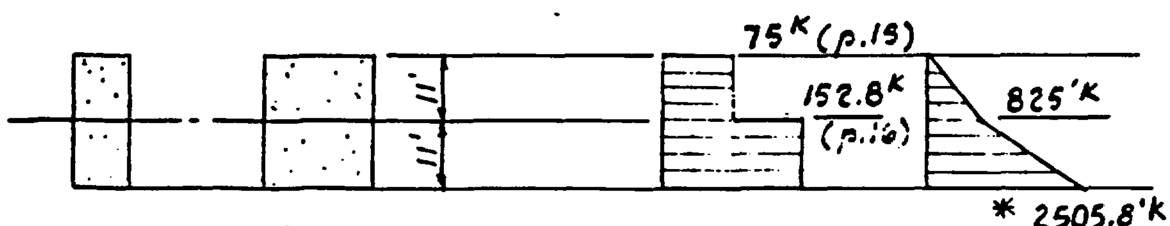
WALL 1



WALL 3 & 5



WALL 7



WALLS A & C

*NOTE: THE OVERTURNING MOMENT IS SLIGHTLY LARGER THAN AS SHOWN ON p 6 ($\frac{4950'^K}{2}$ = 2475) SINCE THE TORSIONAL FORCE IS ASSUMED ADDING TO THE OVERTURNING.

VERTICAL LOAD DESIGN

THESE CALCULATIONS ARE EXTRACTED FROM THE VERTICAL LOAD CALCULATION WHICH ARE REQUIRED FOR COMBINING WITH LATERAL LOADS.

WALL	DEAD LOAD	LIVE LOAD
1	(p. 2) ROOF 24.5 #/□' x 16' = 391 #/' (LESS GIRDER + COL.) WALL 125 #/□' x 12' AVG = 1500 LESS WALL OPEN'G 9' x 5' x 125/48' #/□' = ⟨-117⟩ 1774 #/' 2ND FLR. 73 #/□' x 16' = 1168 (LESS GIRDER + COL.) WALL 125 #/□' x 11 = 1375 LESS OPEN'G = ⟨-117⟩ 4200 #/' FDN. WALL 125# x 1.5' = 188 FTG. (ASSUME 2.5 WIDE) = 563 TOTAL DEAD = 4951 #/'	ROOF 20# x 16' = 320 #/' 2ND FLR. 50# x 16' = 800 TOTAL LIVE = 1120 #/'

FOOTING WIDTH REQ'D = $\frac{4951 + 1120}{3000 \text{ PSF}} = 2.02$ TRY 2'-6" x 18" CONT. FTG.

NOTE: FOOTING WIDTH TO BE CHECKED FOR SEISMIC LOAD

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
7	ROOF (p. 2) 24.5#/□'×16' = 391#/' (LESS GIRDER + COL.) WALL 125#/□'×12' AVG. = 1500#/' 1891#/' 2ND FLR 73#/□'×16' = 1168#/' (LESS GIRDER + COL.) WALL 125#/□'×11' = 1375#/' FDN WALL 125#/□'×1.5' = 188#/' FTG. (ASSUMED 2.75' WIDE = 619#/' TOTAL DEAD = 5241#/'	ROOF 20#×16' = 320#/' 2ND FLR. 50#×16' = 800#/' TOTAL LIVE = 1120#/'

FOOTING WIDTH REQ'D = $\frac{5241 + 1120}{3000\text{ PSF}}$ = 2.12'

TRY, 2'-9"× 18" CONT. FTG.

VERTICAL LOAD DESIGN

WALL	DEAD LOAD	LIVE LOAD
3 (WALL 5 SIM)	(p.2) ROOF 24.5#/□' x 32' = 784#/' WALL 125#/□' x 12' AVG. = 1500 LESS WALL OP'N'G 6' x 5' x 125# x $\frac{4}{48}$' = ⟨-312⟩ 1972 (p. 2) 2ND FLR 73#/□' x 32' = 2336 WALL 125#/□' x 11' = 1375 LESS OPEN'G = ⟨-312⟩ 5371#/' FDN WALL 125#/□' x 1.5' = 188#/' FTG (ASSUME 3' WIDE) = 675#/' TOTAL DEAD = 6234#/'	ROOF 20#/□' x 32' = 640#/' 2ND FLR 50#/□' x 32' = 1600#/' TOTAL LIVE = 2240#/'

FOOTING WIDTH REQ'D = $\frac{6234 + 2240}{3000 psf}$ = 2.82'

TRY 3'x0" x 18" CONT. FTG.

VERTICAL LOAD DESIGN

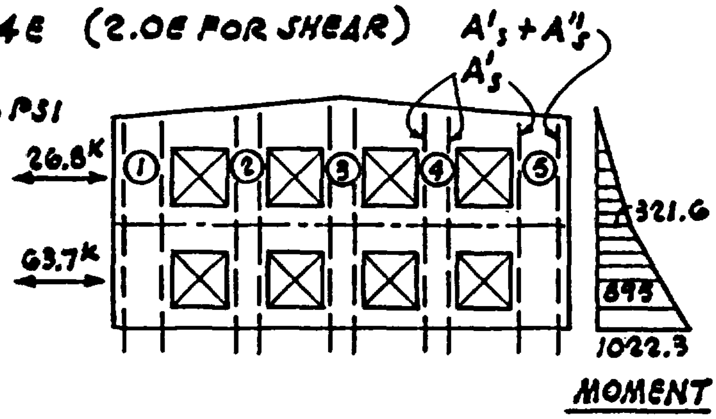
WALL	DEAD LOAD	LIVE LOAD
A (9' PIER) (WALL C SIM.)	(p. 2) ROOF 24.5#/□' x 16' x 3' = 1176# WALL 125#/□' x 9' x 11' = 12375# 13551# $WT/FT = \frac{13551}{9} = 1506\#/'$ 2ND FLR 73.0#/□' x 16' x 3' = 3504# WALL = 12375# $WT/FT = \frac{29430}{9'} = 3270\#/'$ 29430# TOTAL DEAD (EXCL. FTG.) = 29430# ALLOW SOIL PRESS. = 3000 PSF − (300 PSF WT FTG) = 2700 PSF $AREA\ REQ'D = \frac{29430 + 3360}{2700} = 12.1\ \square'$	ROOF 20#/□' x 16' x 3' = 960# 2ND FLR 50#/□' x 16' x 3' = 2400# $WT/FT = \frac{2400}{9'} = 270\#/'$ TOTAL LIVE = 3360# TRY, 8'-0" x 20'-0" FTG. REQ'D FOR SEISMIC OVERTURNING. A = 160 □'
18' PIER	ROOF 24.5#/□' x 32' x 3' = 2352# WALL 125#/□' x 18' x 11' = 24750# 27102# $WT/FT = \frac{27102}{18} = 1506\#/'$ 2ND FLR 73#/□' x 32' x 3' = 7008# WALL = 24750# 58860# $WT/FT = \frac{58860}{18} = 3270\#/'$ TOTAL DEAD (EXCL. FTG) = 58860# ALLOW SOIL PRESS. = 2700 PSF $AREA\ REQ'D = \frac{58860 + 6720}{2700} = 24.3\ \square'$	ROOF 20#/□' x 32' x 3' 1920# 2ND FLR 50#/□' x 32' x 3' = 4800# TOTAL LIVE = 6720# TRY, 8'-0" x 31'-0" FTG. REQ'D FOR SEISMIC OVERTURNING A = 248 □'

Example A-1 21 of 44 Box System

WALL DESIGN

WALL 1

EQU: 6-1 $U = 1.4(D+L) + 1.4E$ (2.0E FOR SHEAR)
EQU: 6-2 $U = 0.9D + 1.4E$
EQU: 6-3 $v = 2\sqrt{f'_c} = 126$ PSI



THE INDIVIDUAL PIERS IN WALL 1 SHOULD BE DESIGNED FOR THE BENDING MOMENT DUE TO THE LATERAL LOAD ($M = V \times h/2$) PLUS THE AXIAL LOADS FROM DEAD AND LIVE LOADS (EXCEPT ROOF L.L.) PLUS THE AXIAL LOAD DUE TO WALL OVERTURNING. UNLESS AXIAL LOADS ARE EXCEPTIONALLY LARGE IT IS USUALLY CONSERVATIVE TO NEGLECT AXIAL LOADS. THIS PROBLEM PROCEEDS ON THIS SIMPLIFYING ASSUMPTION. (SEE PCA BULLETIN EB004.05D FOR AXIAL PLUS BENDING INTERACTION METHOD)

SEE NEXT SHT. FOR SAMPLE CALCULATION

	PIER	WIDTH	R (P.10)	V	A_c	$v = \frac{2V}{\phi A_c}$	$M = V\frac{h}{2}$	A'_s	A''_s	$A'_s + A''_s$	REINF.
1ST STORY	1	72"	11.1	19.8K	720□"	65	59.4	0.40□"	0.73□"	1.13□" 0.53	2-#7 2-#5
	2	48"	4.5	8.0	480	39	24	0.33		0.33	2-#5
	3	48"	4.5	8.0	480	39	24	0.33		0.33	2-#5
	4	48"	4.5	8.0	480	39	24	0.33		0.33	2-#5
	5	72"	11.1	19.8K	720	65	59.4	0.40	0.73	0.53 1.13□"	2-#5 2-#7
			Σ=35.7	Σ=63.7K (P.16)							

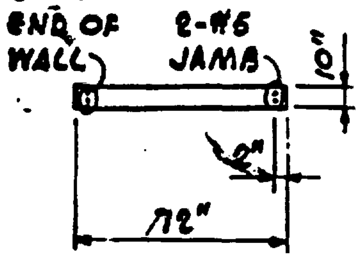
NOTE: 1. A'_s INCREASED BY 1/3 PER ACI § 10.5.2
2. MIN. REINF. 2-#5 PER FIG. 6-8
3. REINF. FOR 1ST STORY PIERS IS EXTENDED TO THE 2ND STORY, THEREFORE CALCULATIONS FOR 2ND STORY ARE NOT INCLUDED

WALL DESIGN (CONT.)

WALL 1

SAMPLE CALCULATION FOR SHT. 22:

1^{ST} STORY - PIER 1



PLAN - PIER 1

SHEAR IN PIER 1

$V_1 = \frac{R}{\Sigma R} \times V_{WALL} = \frac{11.1}{35.7} \times 63.7^K = 19.8^K$ ← (P.16)

$V_u = 2.0 \times V_1 = 39.6^K$ (EQ: 6-1)

$v = \frac{V_u}{\phi A_c} = \frac{39.6}{0.85 \times 720} = 65\ PSI < 126\ PSI$ OK

MOMENT IN PIER 1 DUE TO PIER SHEAR

$M = V_1 \times \frac{h}{2} = 19.8 \times \frac{6'}{2} = 59.4$

$M_u = 1.4M = 1.4 \times 59.4 = 83.2^{IK}$ (EQ: 6-1)

REQ'D $A_s' = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{83.2 \times 12}{0.9 \times 40 \times (70 - \frac{1.0}{2})} = 0.40^{\square''}$

WHERE a IS ASSUMED AS 1.0

CHECK ASSUMPTION:

$a = \frac{A_s f_y}{0.85 f_c' b_w} = \frac{0.40 \times 40}{.85 \times 4 \times 10} = 0.47 < 1.0$ ASSUMED
$\therefore$ OK

OVERTURNING MOMENT OF ENTIRE WALL

THE ENTIRE WALL IS ASSUMED AS A FLEXURAL UNIT FOR DETERMINING REINFORCEMENT TO RESIST OVERTURNING. THIS REINF. IS ADDED AT EACH END OF THE WALL.

$M_{OT} = 895^{IK}$ (P.22)

$M_u = 1.4 \times 895 = 1253^{Ik}$ (EQ: 6-1)

$A_s'' = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{1253 \times 12}{0.9 \times 40 \times (572'' - \frac{1}{2})}$
$= 0.73^{\square''}$

TOTAL $A_s = A_s' + A_s'' = 0.4 + 0.73 = 1.13^{\square''}$ 2-#7
AT EACH END OF WALL

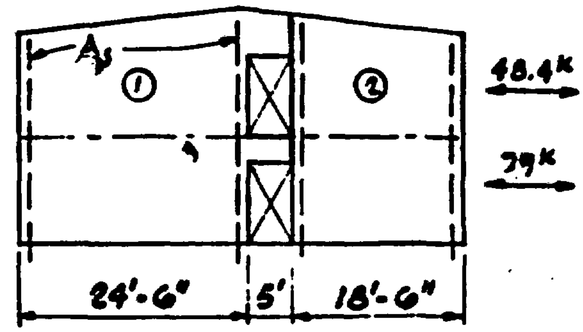
TOTAL $A_s = A_s' = 0.40^{\square''}$ AT EACH JAMB.

ACI ¶10.5.2 SPECIFIES 1/3 INCR. IF $\rho < \frac{200}{f_y}$ $\therefore A_s = 0.4 \times 1.33 = 0.53$ 2-#5

WALL DESIGN

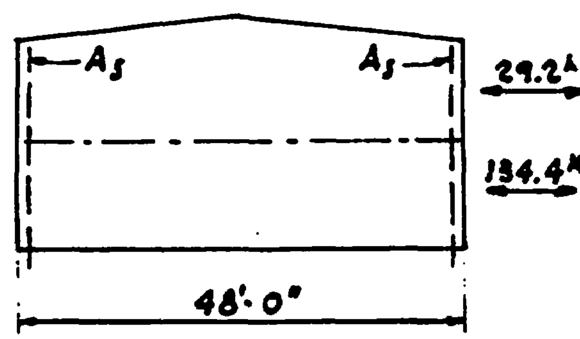
WALL 3 (WALL 5 SIM.)

ASSUME THAT PIERS ACT AS SERIES OF VERTICAL CANTILEVER BEAMS STRUTTED AT ROOF & 2ND FLR. LINE & FIXED AT 1ST FLR.



	PIER	WIDTH	R (P.10)	V	A_c	$v = \frac{2V}{\phi A_c}$	M_{OT}	A_s *	REINF.
1ST STORY	1	24.5'	27	47.5K	2940□"	38 PSI	869'K	1.84□"	2-#9
	2	18.5'	17.9	31.5	2220	33	576'	1.61	2-#8
			Σ=44.9	Σ=79K (P.16)			Σ=1445'K (P.17)		

WALL 7

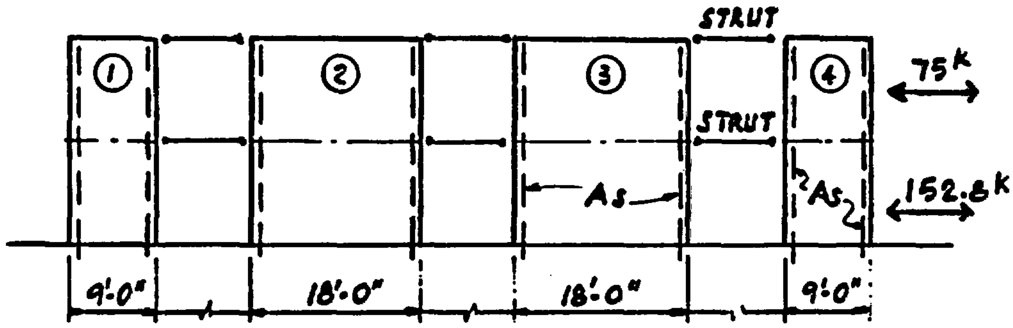


	PIER	WIDTH	R (P.11)	V	A_c	$v = \frac{2V}{\phi A_c}$	M_{OT}	A_s *	REINF.
1ST STORY	1	48'	58.8	134.4K	5760	55 PSI	1828.8 (P.17)	1.97	2-#9

* INCREASE A_s BY 1/3 PER ACI ¶ 10.5.2

WALL DESIGN

WALL A (WALL C SIM.)



	PIER	WIDTH	R (p 11)	V	Ac	$\tau = \frac{2V}{\phi A_c}$	M_{OT}	As	
1ST. STORY	1	9'	4.5	19.3^k	1080	42 psi	316.7	1.82	2-#9
	2	18'	13.3	57.1	2160	62	936.2	2.69	3-#9
	3	18'	13.3	57.1	2160	62	936.2	2.69	3-#9
	4	9'	4.5	19.3	1080	42	316.7	1.82	2-#9
			Σ 35.6	Σ 152.8^k (p 17)			Σ $2505.8'^k$ (p 17)		

SAMPLE CALCULATION FOR PIER 1 :

SHEAR $V = \frac{R}{\Sigma R} V_{WALL} = \frac{4.5}{35.6} \times 152.8^k = 19.3^k$

$V_u = 2V = 2 \times 19.3^k = 38.6^k$ (EQ. G:1)

$\tau = \frac{2V}{\phi A_c} = \frac{V_u}{\phi A_c} = \frac{38.6}{.85 \times 1080} = 42 \text{ psi} < 126 \text{ psi}$ OK

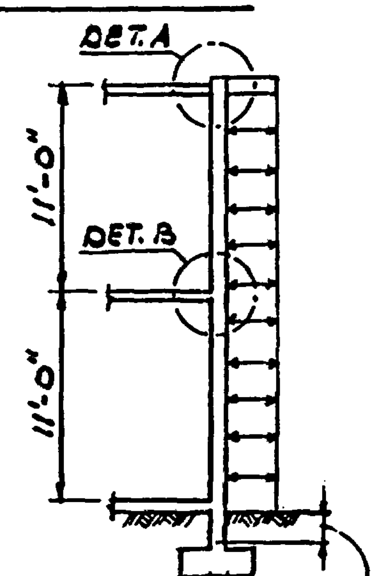
MOMENT $M_1 = \frac{R}{\Sigma R} M_{OT} = \frac{4.5}{35.6} \times 2505.8'^k = 316.7'^k$

$M_u = 1.4 M_1 = 1.4 \times 316.7 = 443.4'^k$

$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{443.4 \times 12}{0.9 \times 40 \times (108'' - \frac{3}{2})} = 1.38^{\square''}$

$A_s = 1.33 \times 1.38 = 1.82^{\square''}$ INCREASE PER ACI # 10.5.2

WALL DESIGN - SEISMIC FORCES NORMAL TO WALL



NOTE: WHEN FLOATING SLAB IS USED, ASSUME PT. OF FIXITY 2' BELOW GRADE LINE FOR DESIGN OF WALL

EQUI 3.8 $F_p = ZIC_pW_p$

WHERE $C_p = 0.30$ (TABLE 3-4)

$W_p = 125^{\#}/\square'$ (10" CONC)

$Z = 1.0 \quad I = 1.0$

$F_p = F_p = 1 \times 1 \times 0.3 \times 125 = 37.5^{\#}/\square'$

REACTION @ ROOF $= 37.5^{\#}/' \times 11'/2 = 206^{\#}/'$

CONT. SPAN

REACTION @ 2ND FLR $= 37.5^{\#}/' \times 11' \times \frac{10}{8} = 516^{\#}/'$

MIM. LOAD $= 200^{\#}/'$ (PARA = 3-3(J)3a)

MAX. WALL BENDING @ 2ND FLR LINE

$$M = \frac{wl^2}{8} + \frac{M_{ecc}}{2}$$

$$= \frac{37.5^{\#} \times 11'^2 \times 12}{8} + \frac{(73^{\#}+50^{\#}) \times 3 \times 7'' ecc.}{2}$$

$$= 8098'^{\#}$$

ASSUME $a = 0.1''$

$$A_s = \frac{M}{\phi f_y (d - \frac{a}{2})} = \frac{8.098''^K}{0.9 \times 40 (8.5 - \frac{0.1}{2})}$$

$$= 0.027^{\square''}$$

CHECK ASSUMED $a = 0.1$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.027 \times 40}{0.85 \times 4 \times 10} = 0.03 < 0.1$$

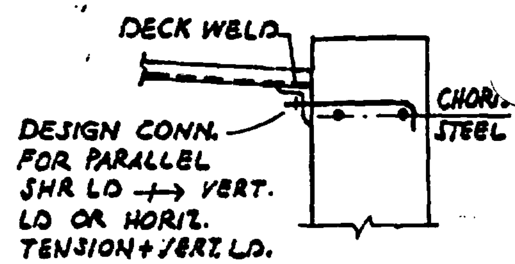
OK

MIN. $A_s = .0025\,bd$ (PARA. 6-3a(1)c)

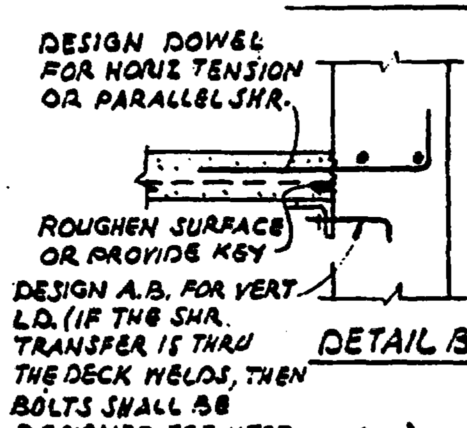
$= .0025 \times 10 \times 8.5 = 0.21^{\square''}$

USE #4 @ 16" O.C. E.F.

$A_s = 0.30$



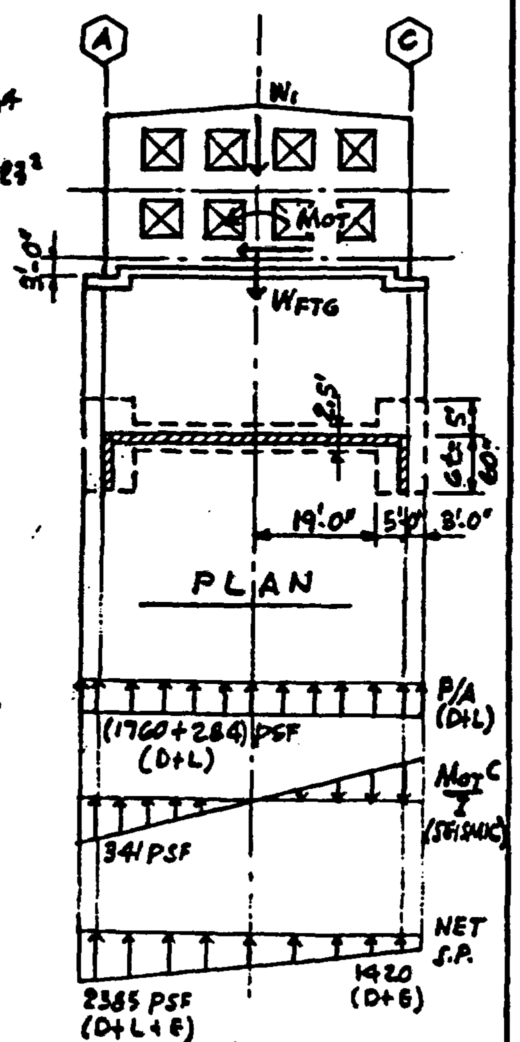
DETAIL A



DETAIL B

FOOTING DESIGN FOR SEISMIC LOADS

WALL 1



MOMENT OF INERTIA OF FTG.:

FOR VERTICAL LOAD $I_1 = 2.5' \times \frac{48^3}{12} = 23040\ FT^4$

FOR SEISMIC LOAD $I_2 = \frac{2.5' \times 38^3}{12} + 2 \times 8' \times 10' \times 23^2$
INCLUDE RETURN WALL FTGS. $= 96072\ FT^4$

AREA OF FTG. = 2.5'×54' = 135 □'

WEIGHT (p.18)

$W_1 = (4200^{\#}/') \times 48' = 201600^{\#}$ (DEAD)

$W_1 = (800^{\#}/') \times 48' = 38400^{\#}$ (LIVE W/O ROOF L.L.)

$W_{FTG} = (751^{\#}/' \times 48' = 36000$

ΣW(DEAD) = 237600

ΣW(LIVE) = 38400

OVERTURNING MOMENT @ BASE OF FTG.

$M_{OT} = \underset{℄(p.17)}{1022.3'^{K}} + \underset{℄(p.17)}{63.7^{K}} \times 3' = 1213.4'^{K}$

SOIL PRESSURE		MAX.	MIN.
P/A (DEAD)	$\frac{237600}{135}$	+1760	+1760
P/A (LIVE)	$\frac{38400}{135}$	+ 184	
$M_{OT}C$	$\frac{1213.4 \times 27'}{96072}$	+ 341	-341
		2385 < 3000 × 1⅓	1420 NO UPLIFT

NOTE: THE SOIL PRESSURE UNDER THE RETURN WALLS DUE TO OVERTURNING (341 PSF) IS ADDED TO THE SOIL PRESSURE UNDER THE RETURN WALL WHICH IS VERY LOW ∴ OK

FOOTING DESIGN FOR SEISMIC LOADS

WALL 7

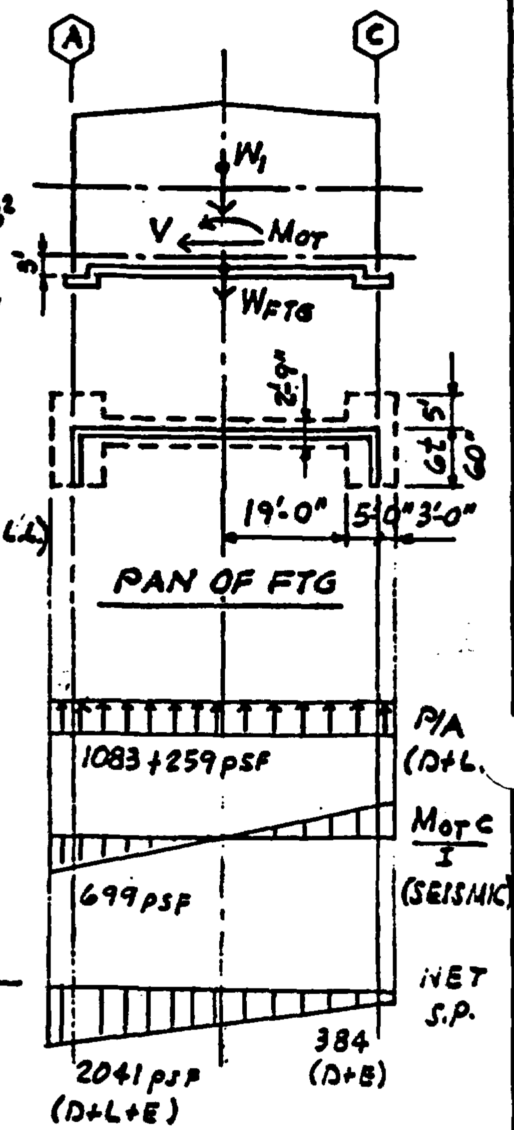
MOMENT OF INERTIA OF FTG:

FOR VERTICAL Ld $I_1 = \frac{2.75 \times 48^3}{12} = 25344\ FT^4$

FOR SEISMIC Ld $I_2 = \frac{2.75 \times 19^3}{12} + 2 \times 8' \times 10' \times 23^2$

$= 86212\ FT^4$

AREA OF FTG = $2.75' \times 54' = 148.5^{\square'}$



WEIGHT (p.19)

$W_1 = (2543^{\#}/' \times 48' = 122064^{\#}$ (DEAD)

$W_1 = (800^{\#}/' \times 48' = 38400$ (LIVE W/o ROOF LL)

$W_{FTG} = (806^{\#}/' \times 48' = 38724$

ΣW (DEAD) $= 160788^{\#}$

ΣW (LIVE) $= 38400$

OVERTURNING MOMENT AT BASE OF FTG.

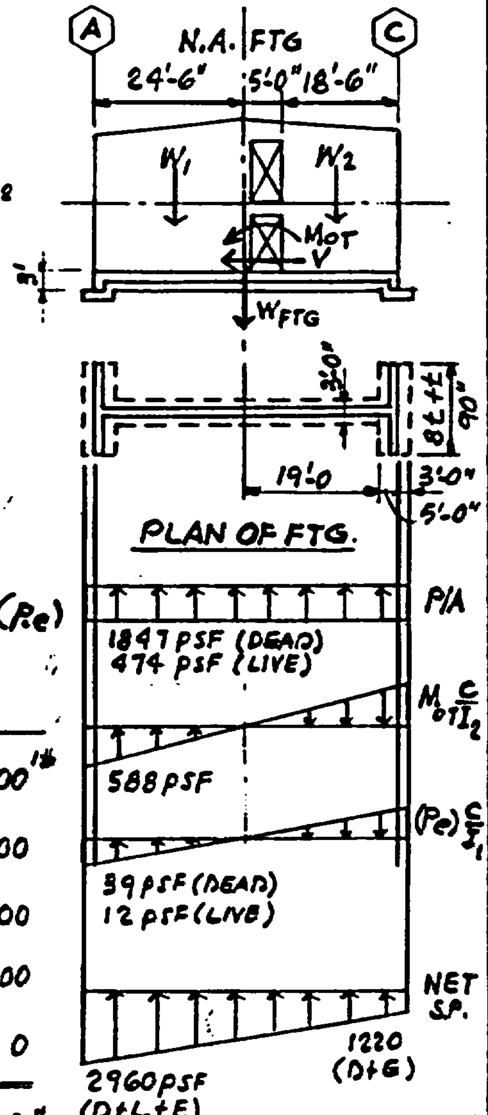
(p.17)

$M_{OT} = 1828.8 + 134.4^k \times 3' = 2232'^k$

SOIL PRESSURE		MAX	MIN.
P/A (DEAD)	$\frac{160788}{148.5}$	+1083	+1083
P/A (LIVE)	$\frac{38400}{148.5}$	+ 259	
$\frac{M_{OT} C}{I_2}$	$\frac{2232 \times 27}{86212}$	+ 699	− 699
		+ 2041 (3000 × 1⅓	+ 384 NO UPLIFT

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (WALL 5 SIM.)



MOMENT OF INERTIA OF FTG:

FOR VERTICAL LD $I_1 = \frac{3 \times 48^3}{12} = 27648\ FT^4$

FOR SEISMIC LD $I_2 = \frac{3 \times 38^3}{12} + 2 \times 8' \times 7.5 \times 23^2$

$= 77200\ FT^4$

AREA OF FTG. $3' \times 54' = 162^{\square'}$

OVERTURN'G MOMENT = $1445^{'k}$ (p.17)

SHEAR V = 79^K (p.17)

OVERTURN'G MOMENT @ BASE OF FTG.

$M_{OT} = 1445^{'K} + 79^K \times 3 = 1682^{'K}$

CALCULATION OF ECCENTRIC MOMENT (Pe) OF WALL MASS RESPECT TO N.A.

WEIGHTS (p.20)		X	DIST. TO N.A. FTG	= WX
$W_{1\,(DEAD)} = 537\#/' \ (24.5' + 2.5')$	$= 14500^\#$	X	$-11.75 =$	$-1703800^{'\#}$
$W_{1\,(LIVE)} = 1600\#/' \ (24.5' + 2.5')$	$= 43200$	X	$-11.75 =$	-507600
$W_{2\,(DEAD)} = 5371\#/' \ (18.5' + 2.5')$	$= 112800$	X	$+14.75 =$	1663800
$W_{2\,(LIVE)} = 1600\#/' \ (18.5' + 2.5')$	$= 33600$	X	$+14.75 =$	495600
$W_{FTG} = 863\#/' \times 48'$	41400	X	$0 =$	0

$\Sigma W\ (DEAD) = 299200^\#$ $\Sigma W(DEAD) - 40000^\#$

$\Sigma W\ (LIVE) = 76800^\#$ $\Sigma W(LIVE) - 12000^\#$

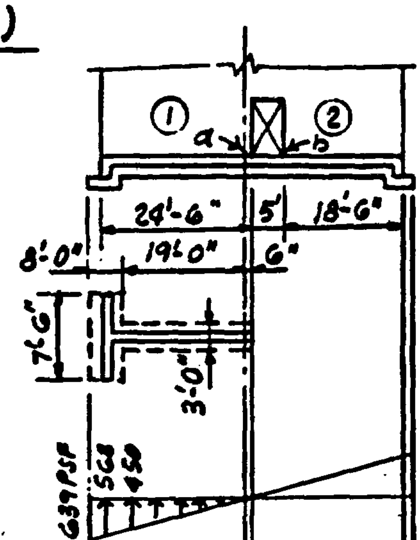
ECCENTRICITY $e = \frac{-40000}{299200} = -0.13'\ (DEAD)$

$e = \frac{-12000}{76800} = -0.156'\ (LIVE)$

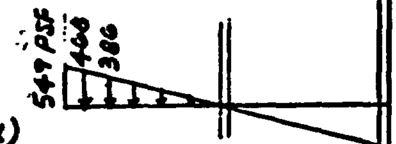
FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (CONT.) (WALL 5 SIM)

SOIL PRESS. PSF	MAX.	MIN.
$\frac{P}{A}$ (DEAD) $\frac{299200}{162}$	$+1847$	$+1847$
$\frac{P}{A}$ (LIVE) $\frac{76800}{162}$	$+474$	
$M_{OT}\frac{C_2}{I_2}$ $\frac{1682\times27}{77200}$	$+588$	-588
$M_{ecc}\frac{C_1}{I_1}$ (DEAD) $\frac{40000\times27}{27648}$	$+39$	-39
$M_{ecc}\frac{C_1}{I_1}$ (LIVE) $\frac{12000\times27}{27648}$	$+12$	
	$2960 <$ $3000\times1\frac{1}{3}$	$+1220$ NO UPLIFT



SOIL PRESS.
(OVERTURN. C. CLKWISE + ECCENTR.CITY OF WTS)



SOIL PRESS.
(OVERTURN. CLKWISE + ECCENTRICITY OF WTS)

CHECK SHEAR IN FDN. WALL @ DOOR OP'NG (Pt. a)

OVERTURNING COUNTER CLOCKWISE $\Sigma V=0$

$V=(450\times\frac{19'}{2}\times3'_{WIDE})+(568\times8'\times7.5)=46.9^{K}$

$v_u=\frac{2V}{\phi bd}=\frac{2\times469}{0.85\times10\times32}=345$

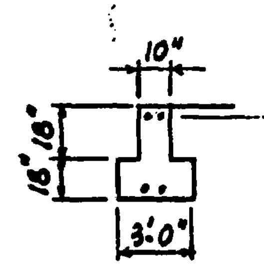
$v_c=2\sqrt{f'_c}=2\sqrt{4000}=126$

$v_u-v_c=345-126=219\,psi$ (PROVIDE STIRRUPS)

MAX $(v_u-v_c=8\sqrt{f'_c}=504\,psi$ (ACI # 11.6)

OVERTURNING CLOCKWISE $\Sigma V=0$

$V=(386\times\frac{19}{2}\times3'_{WIDE})+(468\times8'\times7.5')$
$=39^{K}<46.9$ COUNTER CLOCKWISE IS MORE CRITICAL.

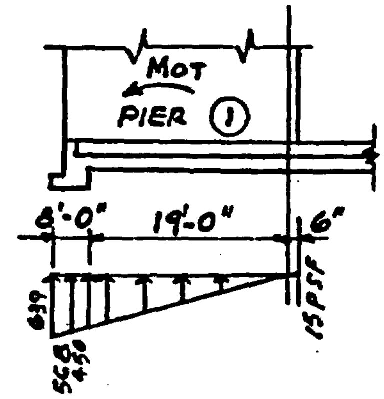


SECTION

FOOTING DESIGN FOR SEISMIC LOADS

WALL 3 (CONT.)

CHECK MOMENT IN FDN WALL @ DOOR OP'N'G (Pt. a)



$M_{OT}(\text{PIER }1) = \Sigma M_{OT}$

$= \frac{27}{44.9} \times 1682'^{K} = 1011'^{K}$

MOMENT AT Pt. a

$M_a = 1011'^{K} - \left[(450 \times \frac{19}{2} \times 3'\text{WIDE}) \times (19' \times \frac{2}{3} + 0.5')\right]$

$- \left[568 \times 7.5' \times 8' \times 23'\right] = 58.2^{K}$

$F = \frac{bd^2}{12000} = \frac{10 \times 32^2}{12000} = 0.86$

$K = \frac{M_u}{F} = \frac{1.4 \times 58.2}{0.86} = 96$

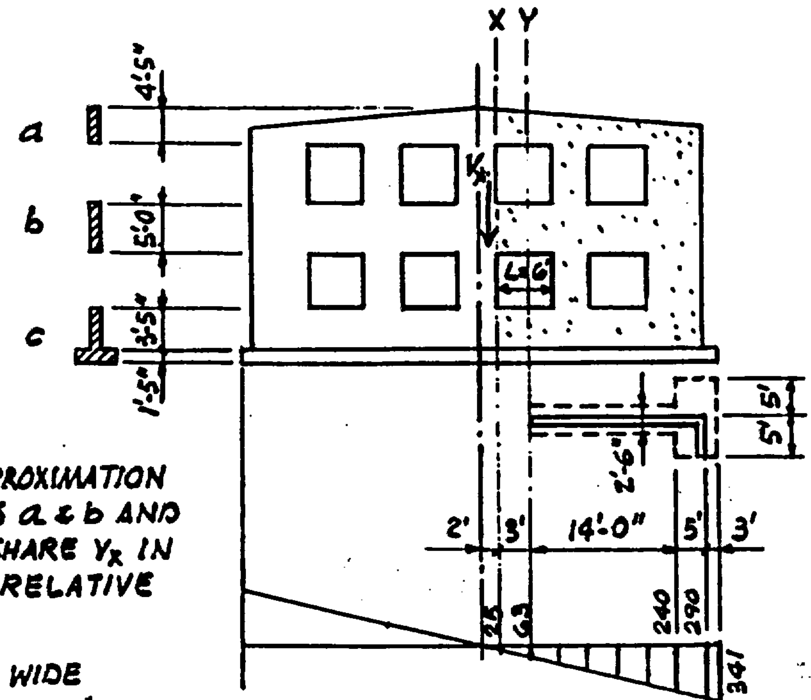
$a_u = 2.96$ (SP-17 FLEX 1.2)

$A_s = \frac{M_u}{a_u d} = \frac{1.4 \times 58.2}{2.96 \times 32} = 0.86^{\square''}$ 2-#7 TOP & BOTT

CHECK OPPOSITE FACE OF OPENING IN SIMILAR MANNER

SPANDREL DESIGN

WALL 1



SOIL PRESSURE (SEISMIC)
(p.27)

AS A SIMPLYFYING APPROXIMATION ASSUME THAT SPANDRELS a & b AND FOUNDATION WALL c SHARE V_X IN PROPORTION TO THEIR RELATIVE RIGIDITIES

$V_x = \frac{240+25}{2} \times 17' \times 2.5$ WIDE $+ 290 \times 10' \times 8' = 28831^{\#}$ SHEAR DUE TO SEISMIC OVERTURNING

$V_y = \frac{240+63}{2} \times 14 \times 2.5 + 290 \times 8 \times 10 = 28502^{\#}$

	$I (FT^4)$	L	$R = I/L$	$R/\Sigma R$	V_x	$v = \frac{2V}{\phi A_c}$	V_y	$M = \frac{VL}{2}$	W_{D+L}	$M' = \frac{wl^2}{12}$	$M+M'$
4.5'	6.3	6'	1.05	0.22	6.3^K	27psi	6.3	$18.9^{'K}$	$954^{\#}/'$	$2.86^{'K}$	$21.9^{'K}$
5'	8.68	6'	1.45	0.31	8.9	35	8.8	26.4	$2593^{\#}/'$	$7.78^{'K}$	$34.2^{'K}$
5' 2.5'	13.31	6'	2.22	0.47	13.5	53	13.4	40.2	$1170^{\#}/'$	$3.5^{'K}$	$43.7^{'K}$
			4.72	1.0	28.8^K		28.5^K				

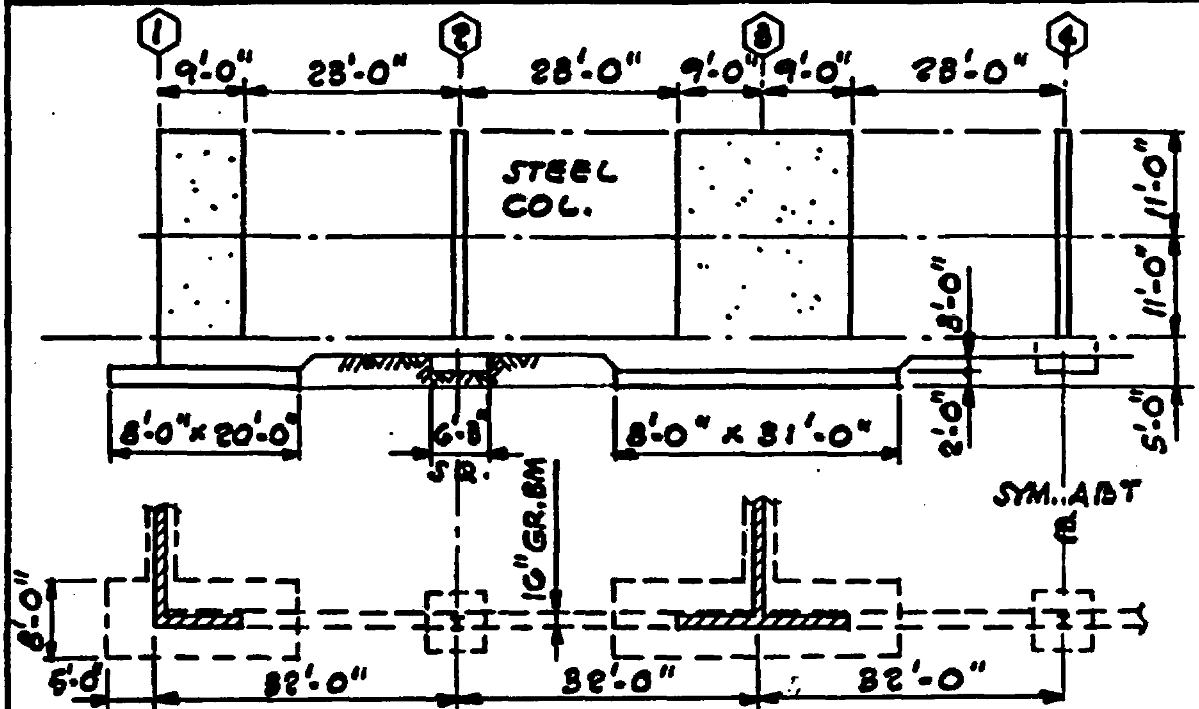
DESIGN SPANDREL FOR MAX. MOMENT $(M+M')$

SAMPLE CALCULATION: $V_x = 28.8^K \times R/\Sigma R = 28.8^K \times .22 = 6.3^K$

$W_{D+L} = (391^{\#}/') + 4.5' \times 125^{\#} = 954^{\#}/'$ UNIT WT. ON SPANDREL

℄ (p.18)

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM.)



PLAN OF FOOTINGS

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM.)

18' PIER

TOTAL WALL OVERTURN'G MOMENT = 2506.8'K (P.17)

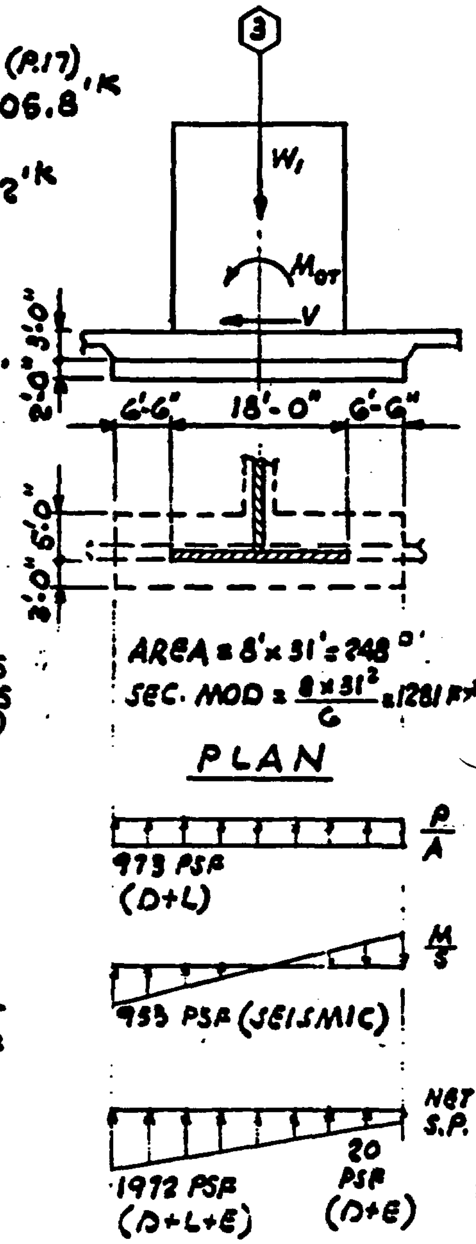
OVERTURN'G MOMENT TO 18' PIER =

$\frac{R}{\Sigma R} \times 1628'^K = \frac{13.3}{35.6} \times 2505.8 = 936.2'^K$ (P.11)

SHEAR $V = 57.1^K$ (P.25)

OVERTURN'G MOMENT @ BASE OF FTG.:

$M_{OT} = 936.2'^K + 57.1' \times 5' = 1221.7'^K$



WEIGHTS

W_1 = 58860# (DEAD) (P.21)
W_1 = 4800#(LIVE EXCL. ROOF L.L.) (P.21)
W_1 = 5371 #/ × 4.17' TRIB. = 22400#(DEAD) } CROSS WALLS (P.20)
W_1 = 1600#/ × 4.17' TRIB = 6672#(LIVE) }
ΣW_1 (DEAD) = 81260 #
ΣW_1 (LIVE) = 11472 #
W_{FTG} = 2' × 150# = 300 #/□'
W_{SOIL} = 3' × 115# = 345 #/□'

SOIL PRESSURE	MAX.	MIN.
P/A (FTG + SOIL)	+645 PSF	+645 PSF
P/A (DEAD) $\frac{81260}{248}$	+ 328	+328
P/A (LIVE) $\frac{11472}{248}$	+ 46	
M/S (SEISMIC) $\frac{1221,700}{1281}$	+ 953	- 953
	+ 1972 PSF	+ 20 PSF
		NO UPLIFT

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM.)

9' PIER

TOTAL WALL OVERTURN MOMENT = 2505.8'K (P.17)

OVERTURN MOM. TO 9' PIER

$\frac{R}{\Sigma R} \times 2505.8 = \frac{4.5}{35.6} \times 2505.8 = 316.7'^K$ (P.11)

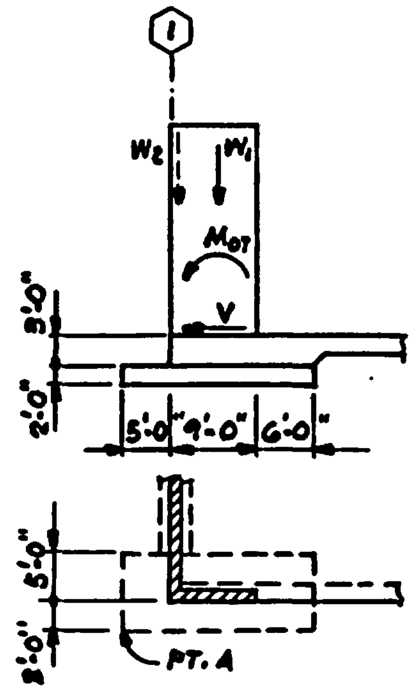
SHEAR $V = 19.3^K$ (P.25)

OVERTURN MOM. @ BASE OF FTG.

$M_{OT} = 316.7 + 19.3^K \times 5' = 413.2'^K$

AREA OF FTG. = 8' × 20' = 160 □'

SECTION MODULUS = $\frac{8 \times 20^2}{6} = 533 FT^3$



PLAN

WEIGHTS	×	DIST. TO PT. A	=	Wd
W_1 (DEAD) = 29480# (P.21)	×	9.5'	=	279585
W_1 (LIVE) = 2400# (P.21)	×	9.5'	=	22800
W_2 (DEAD) = 4200#/' × 4.17' TRIB. = 17514#	×	5.42'	=	94926
W_2 (LIVE) = 800#/' × 4.17' TRIB. = 3336#	×	5.42'	=	18081
$W_{FTG.}$ 2' × 150# × 8' × 20' = 48000#	×	10'	=	480000
W_{SOIL} 3' × 115# × 8' × 20' = 55200#	×	10'	=	552000

$\Sigma W (DEAD) = 150144\#$ $\Sigma Wd (DEAD) = 1406511$

$\Sigma W (LIVE) = 5736\#$ $\Sigma Wd (LIVE) = 40881$

ECCENTRICITY $e (DEAD) = \frac{1406511}{150144} - 10' = 0.63'$

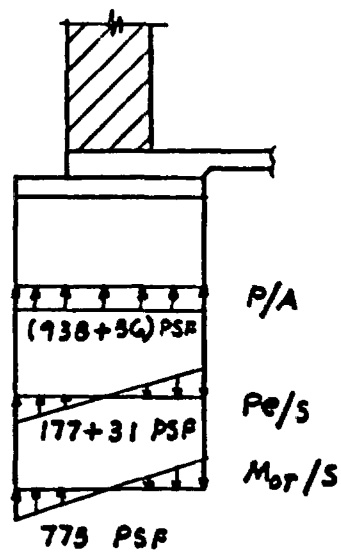
$e (LIVE) = \frac{40881}{5736} - 10' = 2.87'$

FOOTING DESIGN FOR SEISMIC LOADS
WALL A (WALL C SIM)

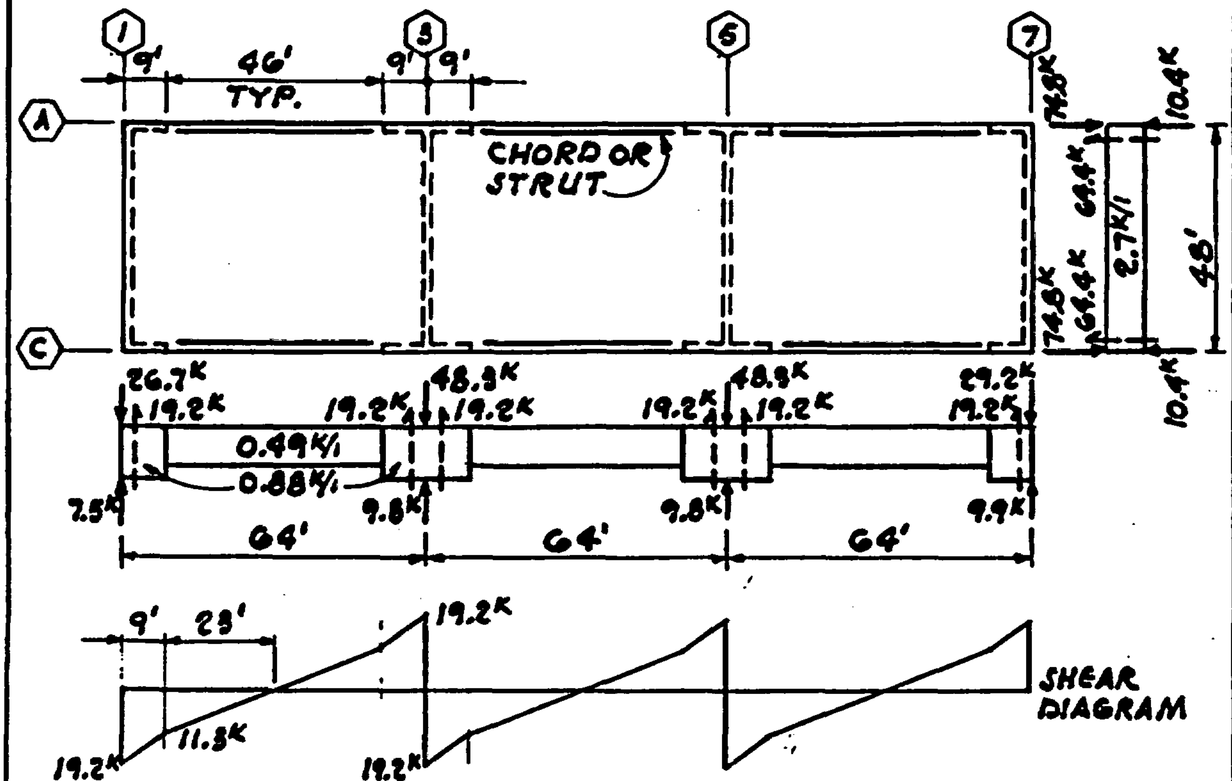
9' PIER (CONT.)

SOIL PRESSURE		MAX	MIN.
P/A (FTG + SOIL)	300# + 345#	= +645 PSF	+645
P/A (DEAD)	$\frac{29430 + 17514}{160}$	= +293	+293
P/A (LIVE)	$\frac{2400 + 3336}{160}$	= +36	
Pe/S (DEAD)	$\frac{150144 \times 0.63'}{533}$	= +177	−177
Pe/S (LIVE)	$\frac{5736 \times 2.87'}{533}$	= +31	
M_{OT}/S (SEISMIC)	$\frac{413,200}{533}$	= +775	−775
		1957 PSF < 3000 × 1⅓ OK	−14 UPLIFT.

SMALL UPLIFT CONSIDERED OK, SINCE GR. BM. CAN OFFER SOME RESISTANCE



DESIGN OF ROOF DIAPHRAGM



EQUATION (3-9): $F_{px} = \left(\frac{F_t + \Sigma F_i}{\Sigma w_i}\right) w_{px}$

$= \left(\frac{0+150}{534}\right) w_{px} = 0.28\, w_{px}$ (E-W DIRECTION)

$= \left(\frac{0+153}{534}\right) w_{px} = 0.286\, w_{px}$ (N-S DIRECTION)

EQUATION (3-9A) MIN. $F_{px} = 0.14ZIw_{px} = 0.14w_{px}$ WHERE $Z=1.0$

MAX. F_{px} NEED NOT EXCEED $0.30ZIw_{px}$ $I=1.0$

FROM DIAGRAM p.4, MULTIPLY ALL VALUES SHOWN @ 100% G BY $C_p = 0.28$ (E-W) & $C_p = 0.286$ (N-S)

MAX. AVE. DIAPH. SHEAR: (N-S) $\frac{19200\#}{48'} = 400\#/'$

(E-W) $\frac{64400\#}{192'} = 335\#/'$

USE 1½" STEEL DECK 20 GA. SPAN 6'-0" ALLOW SHR = 470#/'
5 END WELDS & BUTTON PUNCH @ 24" O.C. (FIG. 5-19)

DESIGN OF ROOF DIAPHRAGM - CONT.

MAXIMUM MOMENT $= \left(\frac{19.2+11.3}{2}\right)9' + \left(11.3 \times \frac{23}{3}\right) = 267'^{K}$

CHORD STRESS (N-S) $= \frac{M}{D} = \frac{267}{47.2'} = 5.7^{K}$

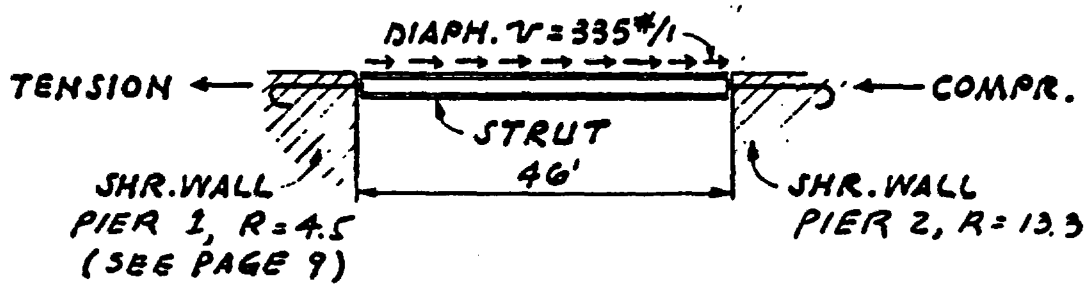
CHORD STRESS (E-W) $= \frac{VL}{4D} = \frac{64.4 \times 48}{4 \times 191.58} = 4^{K}$

DESIGN CHORD FOR TENSION OR COMPR OF 5.7^{K}
DESIGN FOR CHORD REBAR IN WALLS 1 & 7

$A_s = \frac{1.4T}{\phi f_y} = \frac{1.4 \times 4^{K}}{0.9 \times 40} = 0.16\square''$

USE 2-#5

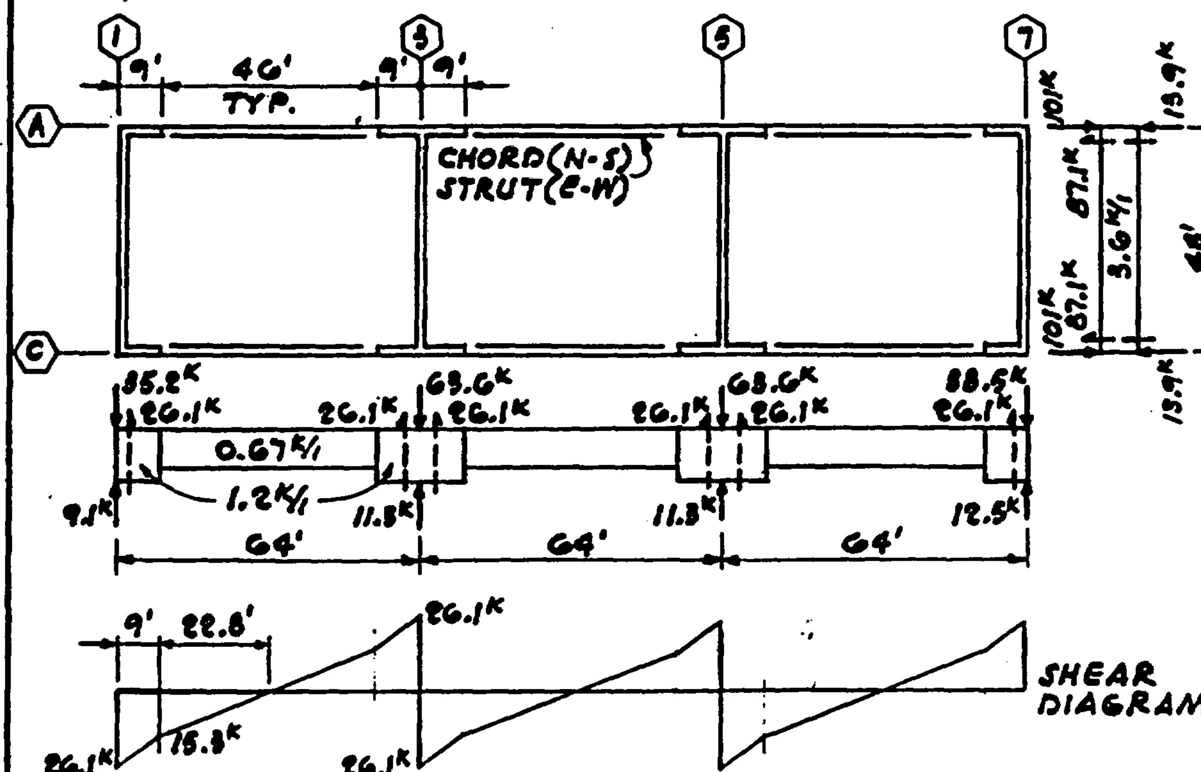
STRUT DESIGN (E-W): IN THE EAST-WEST DIRECTION, THE CHORD BEAMS ALONG (A) & (C) ACT AS COLLECTOR OR DRAG STRUTS. BECAUSE OF WALL RIGIDITIES, DIAPH. SHR. IS TAKEN THRU THE STRUT IN TENSION & COMPR. IN PROPORTION TO THE WALL RIGIDITIES.



$\therefore$ DESIGN STRUT FOR:
$C = T = 335^{\#}/' \times 46'$ LENGTH $\times \frac{13.3}{17.8} = 11{,}514^{\#}$

DESIGN OF 2ND FLOOR DIAPHRAGM

THE CONCRETE DIAPHRAGM SHALL BE DESIGNED TO SATISFY EQUATION 3-9, BUT SHALL NOT BE LESS THAN THAT REQUIRED TO TRANSFER SHEAR ON P.16.



EQUATION (3-9): $F_{px} = \left(\frac{F_t + \Sigma F_i}{\Sigma w_i}\right) w_{px}$

$= \left(\frac{0+300}{1614}\right) w_{px} = 0.186\, w_{px}$ (NORTH-SOUTH & EAST-WEST)

(P.6 & 7)

MIN. $F_{px} = 0.14\, w_{px}$

FROM DIAGRAM P.5, MULTIPLY ALL VALUES SHOWN @ 100 G BY $C_p = 0.186$.

MAX. AVG. DIAPH. SHEAR : (N-S) $\frac{26100\#}{48'} = 544\#/'$

(E-W) $\frac{87100\#}{192} = 454\#/'$

USE 2½" CONC. ON 16-18 GA. STEEL DECK ALLOW. $q = 2770\#/'$
(EQ. 5-27 & FIG 5-31) $f_c' = 3{,}000$ PSI

DESIGN OF 2ND FLOOR DIAPHRAGM (CONT.)

$$\text{MAX. MOMENT} = \left(\frac{26.3^{K} + 15.5^{K}}{2}\right) 9' + \left(15.5^{K} \times \frac{23'}{2}\right) = 366'^{K}$$

$$\text{CHORD STRESS (N-S)} = \frac{M}{D} = \frac{366'^{K}}{47.2} = 7.8^{K}$$

DESIGN CHORD FOR TENSION OR COMPR. OF 7.8^{K}

$$\text{CHORD STRESS (E-W)} = \frac{VL}{4D} = \frac{84.5^{K} \times 48'}{4 \times 191.58} = 5.3^{K}$$

DESIGN FOR CHORD REBAR IN WALLS 1 & 7

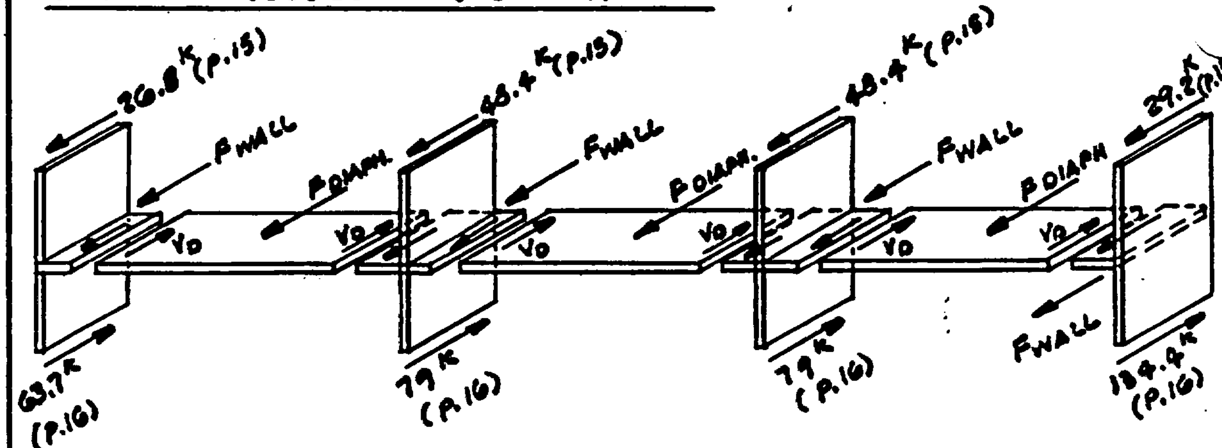
$$A_s = \frac{1.4T}{\phi f_y} = \frac{1.4 \times 5.3}{0.9 \times 40} = 0.21\ \square''$$

USE 2-#5

STRUT DESIGN (E-W):

DESIGN STRUT FOR TENSION & COMP. OF $T = C = 366\ {}^{\#}/' \times \frac{46'}{2} = 8420^{\#}$

CHECK STRESSES FOR SHEAR P.16



WALL 1 WALL 3 WALL 5 WALL 7

V_D = DIAPHRAGM SHEAR

F_{WALL} = SEISMIC FORCE FROM WT OF TRIBUTARY WALL

F_{DIAPH} = SEISMIC FORCE FROM WT OF DIAPHRAGM

DESIGN OF 2ND FLOOR DIAPH. (CONT)

NORTH-SOUTH

WALL 1

$F_{WALL} = 0.186 \times$ TRIB. WALL WT.
$= \underset{℄(p.7)}{0.186} \times \underset{℄(p.5)}{48.6^k} = 9^k$

SHEAR IN WALL ABOVE DIAPH. $= 26.8^k$ (p.15)

SHEAR IN WALL BELOW DIAPH. $= 63.7^k$ (p.16)

DIAPH. SHR $V_D = 63.7^k - 9^k - 26.8^k = 27.9^k$

SHEAR STRESS $v = \dfrac{27900}{48'} = 581^{\#}$ OK

WALL 3

$F_{WALL} = 0.186 \times \overset{℄(p.5)}{60.6^k} = 11.3^k$

SHEAR IN WALL ABOVE DIAPH. $= 48.4^k$ (p.15)

SHEAR IN WALL BELOW DIAPH. $= 79^k$ (p.16)

$F_{DIAPH} = 0.186\left[3.58^{k/\prime} \times 46' + 6.34^{k/\prime} \times 18'\right] = 51.8^k$ (p.5)

DIAPH SHEAR V_D (WEST) $= 51.8^k - 27.9^k = 23.9^k$

DIAPH SHEAR V_D (EAST) $= 79^k - 23.9^k - 48.4^k - 11.3^k = -4.6^k$

SHEAR STRESS $= \dfrac{23900^{\#}}{(48' - 12')} = 664^{\#}/'$ OK
℄ STAIR OPN'G.

WALL 7

$F_{WALL} = 0.186 \times \overset{℄(p.5)}{66.6^k} = 12.4^k$

SHEAR IN WALL ABOVE DIAPH. $= 29.2^k$ (p.15)

SHEAR IN WALL BELOW DIAPH. $= 134.4^k$ (p.16)

DIAPH SHR $V_D = 134.4^k - 29.2^k - 12.4^k = 92.8^k$

SHEAR STRESS $= \dfrac{92800^{\#}}{48'} = 1933^{\#}/' < 2126^{\#}/'$ OK

DESIGN OF 2ND. FLOOR DIAPH. (CONT.)

NORTH-SOUTH

WALL 5

$F_{WALL} = 0.186 \times 60.6^k = 11.3^k$

SHEAR IN WALL ABOVE DIAPH. $= 48.4^k$ (p.15)

SHEAR IN WALL BELOW DIAPH. $= 79^k$ (p.16)

$F_{DIAPH} = 0.186[3.58^{k/'} \times 46' + 6.34^{k/'} \times 18'] = 51.8^k$

DIAPH SHR V_D(EAST) $= 51.8^k - 92.8^k = -41.^k$

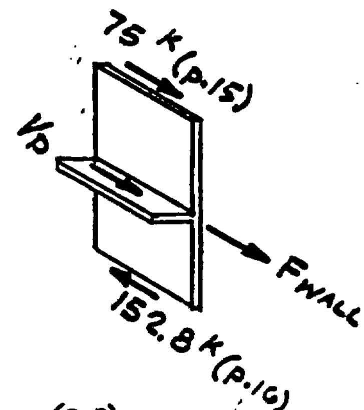
DIAPH SHR V_D(WEST) $= 79^k + 41^k - 11.3^k - 48.4^k - 51.8^k$
$= 8.5^k$

SHEAR STRESS $= \frac{41000}{(48'-12')} = 1139^{\#/'}$ OK

STAIR OP'G.

EAST-WEST

WALL A & C



(p.6) (p.5)

$F_{WALL} = 0.186 \times 74.4^k = 13.8^k$

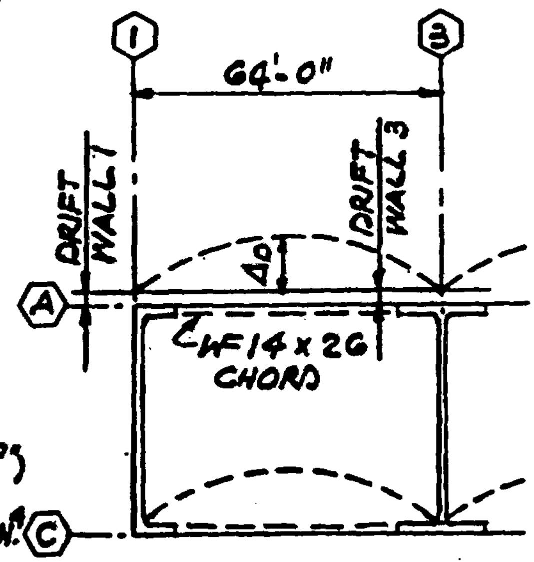
SHEAR IN WALL ABOVE DIAPH. $= 75^k$ (p.15)

SHEAR IN WALL BELOW DIAPH. $= 152.8^k$ (p.16)

DIAPH SHR $V_D = 152.8 - 75^k - 13.8^k = 64^k$

SHEAR STRESS $= \frac{64000}{192} = 333^{\#/'}$ OK

DIAPHRAGM DEFECTION



CHECK DEFLECTION OF ROOF DIAPH. BETWEEN GRID 1 & 3

$\Delta_D = \Delta_{\text{BENDING OF FLANGE}} + \Delta_{\text{SHEAR IN WEB}}$

ASSUME Δ_B IS DEFLECTION OF A SIMPLY SUPPORTED DIAPH.

$$\Delta_B = \frac{5}{384} \cdot \frac{wl^4}{EI}$$

WHERE I IS ASSUMED TO BE BASED ON WF14 x 26 CHORD ($A = 7.67^{\square''}$)

$$I = 2 \times 7.67^{\square''} \times \left(\frac{47.2' \times 12''/'}{2}\right)^2 = 1{,}230{,}300 \text{ IN}^4$$

(p.37)

$$\Delta_B = \frac{5 \times 490^{\#}/' \times 64^4 \times 1728}{384 \times 29 \times 10^6 \times 1230300} = 0.005''$$

(p.37)

AVG. SHEAR/F OF DIAPH $q_{AVG.} = \dfrac{19.2 + 0}{2 \times 48} = 0.200^{K}/'$

FLEXIBILITY $F = 16 + 26.8R$ (SEE FIG. 5-19)

WHERE $R = 6/18 = 0.33$

$F = 16 + 26.8(.33) = 24.8$

DIAPH DEFLECTION FROM SHEAR IN WEB:

$$\Delta_W = \frac{q_{AVG.} L_1 F}{10^6} = \frac{200 \times 82 \times 24.8}{10^6} = 0.159'' \text{ (Equ. 5-2)}$$

TOTAL DIAPH DEFLECTION $\Delta_D = \Delta_B + \Delta_W$

$= 0.005 + .159 = 0.164''$

DRIFT OF SHEAR WALL 1. $\Delta_1 = 0.038'' \times \dfrac{10''}{12''} \times \dfrac{26.8^{K}}{1000^{K}} \times \dfrac{3}{3.6} = 0.00071''$

(p.8) (p.15)

ADJUSTMENT TO FIG C-11 FOR THICKNESS, FORCE & MODU. ELAS.

DRIFT OF SHEAR WALL 3 $\Delta_3 = \left(\dfrac{1}{38.1}\right) \times \dfrac{10''}{12} \times \dfrac{48.4}{1000} \times \dfrac{3}{3.6} = 0.00088''$

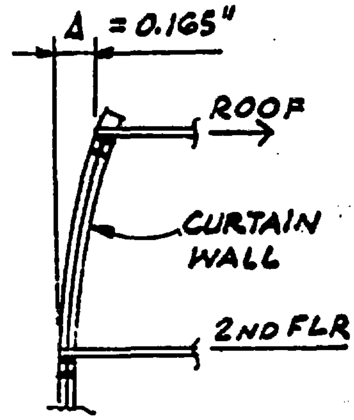
ALLOWABLE DRIFT $= 0.005H = 0.005 \times 12'_{AVG} \times 12 = 0.72'') 0.00088 \times \frac{1}{K}$ (PARA 3-3H)

DIAPHRAGM DEFLECTION

THE AVERAGE DRIFT OF WALL ① & ③ $= \frac{0.00071 + 0.00088}{2} = 0.0008''$

TOTAL RELATIVE DISPLACEMENT OF ROOF DIAPH W/RESPECT TO THE 2ND FLOOR $= 0.164'' + 0.0008$
$= 0.165''$

THE WALL ELEMENT MUST BE DESIGNED TO ACCOMODATE THIS RELATIVE DISPLACEMENT. IN THIS EXAMPLE PROBLEM, THE WALL ELEMENT IS A RELATIVELY FLEXIBLE CURTAIN WALL WHICH PRESENTS NO PROBLEM. THE DEFLECTION CACULATIONS HAVE BEEN INCLUDED PRIMARILY TO ILLUSTRATE THE PROCEDURE IN CASES WHERE BRITTLE WALLS (MASONRY OR CONC.) OCCUR.



DESIGN EXAMPLE: A-2

BUILDING WITH A CONCRETE DUCTILE MOMENT-RESISTING SPACE FRAME:

Description of Structure. A three-story Administration Building with a ductile moment resisting space frame in reinforced concrete without shear walls, using non-bearing, non-shear, exterior walls (skin) of flexible insulated metal panels. The structural concept is illustrated on Sheet 3.

Construction Outline.

Roof:
- Built-up 5-ply.
- Concrete joists and girders.
- Suspended ceiling.

2nd & 3rd Floors:
- Concrete joists and girders.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Non-bearing, non-shear, insulated metal panels.

Partitions:
- Non-structural removable drywall.

Design Concept. Since the structure is a ductile moment resisting space frame with the capacity to resist the total required lateral force, the K-factor is 0.67. Seismic Zone 4. Conc. Frame Type A (Table 3-7).

Discussion. Inasmuch as the design requirements for concrete ductile moment-resisting frames are complex, a detailed design procedure is given on p. 2 of the example.

Loads.

Roof:

5-ply roofing	6.0
1" insulation	1.5
Conc. frame	115.0
Ceiling	5.0
Miscellaneous	3.5
Dead Load	131 psf
Add for seismic loading:	
Partitions	10
	141 psf
Live Load	20 psf

Floors:

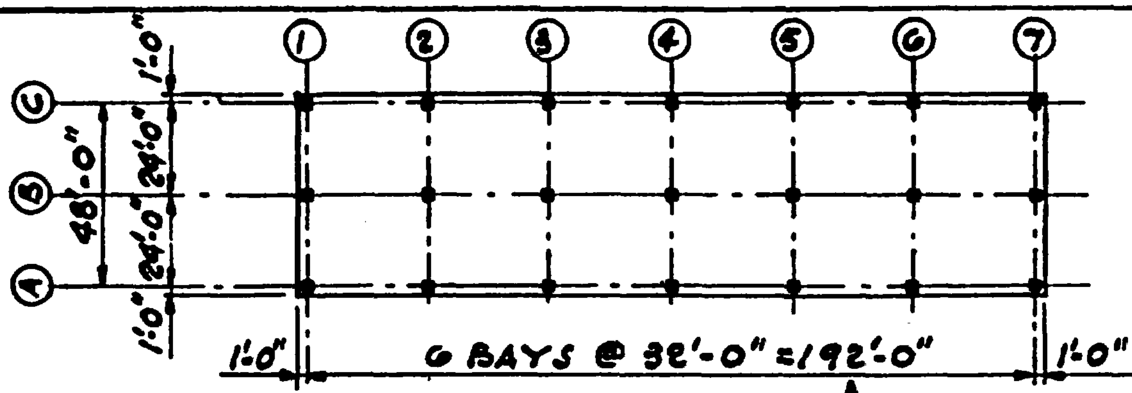
Floor covering	1
Conc. frame	129
Partitions	20
Ceiling	5
Mech. & Elect.	5
Miscellaneous	4
Dead Load	164 psf
Live Load	50 psf
Exterior Wall	4 psf

Materials.

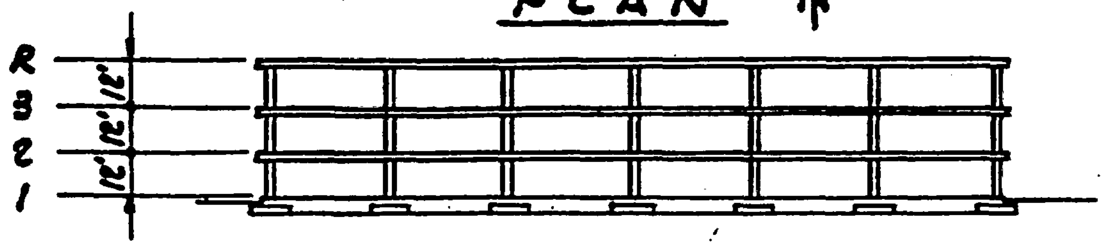
Concrete: $f'_c = 4$ ksi $\quad E_c = 3.6 \times 10^6$ psi

Steel: $f_y = 60{,}000$ psi

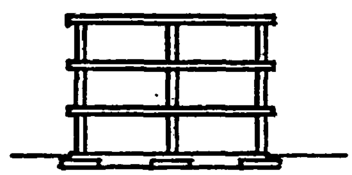
DESIGN PROCEDURE



PLAN



NORTH & SOUTH ELEVATIONS
(SECTION AT LINE B SIMILAR)



EAST & WEST ELEVATIONS
(TRANSVERSE SECTIONS SIMILAR)

SCOPE OF DESIGN EXAMPLE

1. CALCULATION OF LATERAL FORCES ON BLDG. AND DISTRIBUTION TO FRAME
2. TABULATION OF THE RESULTS OF ANALYSIS OF THE FRAME ON LINE B
3. DESIGN OF SECOND FLOOR BEAMS BETWEEN 1 AND 2
4. DESIGN OF COLUMNS B1 & B2

DISCUSSION OF MEMBER SIZES

1. The example is intended to illustrate the procedure for designing a concrete ductile moment resisting frame. The design work is complex, and several trials are required in order to achieve the optimum design.

2. The building configuration was arbitrarily made the same as that of the steel frame of example A-3.

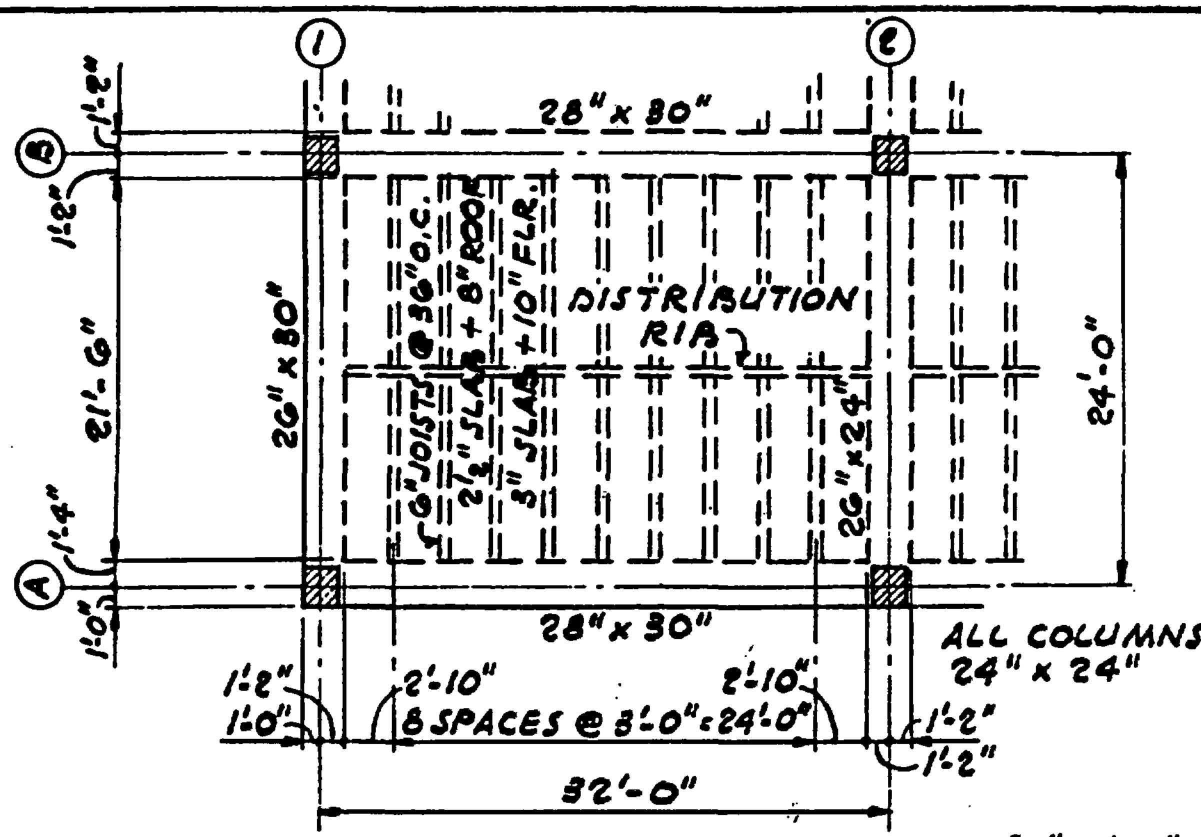
3. Frame B will be analyzed in this example and members between grid lines 1 & 2 will be designed to illustrate the design procedure.

   a. The section of beam & col. sizes is a trial and error procedure. Architectural considerations, limitations on dimensions (Fig. 7-2), space for bar placement, allowable stresses of concrete and steel, etc., can affect the member sizes.

   b. The beam was assumed to be 28" x 30", and the required reinforcing and the actual ultimate moment capacity were calculated.

   c. For the min or max Pu and the required M_p (on the basis of column M_p _ beam M_p), a suitable column was estimated to be 24" x 24", with 12 - #10 or 10 - #11. (Note: Biaxial loading must be considered for column forces in the transverse direction:)

4. Results of a frame analysis are given, and the example continues with representative beam, column and joint design, using sizes and design forces from this analysis. The frame analysis itself is not shown since values can be obtained by computer or by any of the various approximate methods.



STRUCTURAL PLAN SCALE: $\frac{3}{32}" = 1'\text{-}0"$

WEIGHT OF CONCRETE IN TYPICAL 32'x50' BAY

ROOF:

LONGIT. GIRDERS	$3 \times 2.33' \times 2.5' \times 32' \times 0.150$	= 84.0
TRANSV. BEAM	$2 \times 2.17 \times 2.0' \times 21.5 \times 0.150$	= 28.0
JOISTS	$2 \times 29.67 \times 21.5 \times 0.050$	= 63.8
COLUMNS 24x24	$3 \times (9.5/2) \times (2.0)^2 \times 0.150$	= 8.6
		184.4^K

$$\omega_R = \frac{184{,}400^{\#}}{32.0' \times 50.0'} = 115 \text{ PSF}$$

FLOOR:

$$84.0 + 28.0 + \frac{61}{50}(63.8) + 2(8.6) = 207^K \text{ OR } 129 \text{ PSF}$$

BUILDING WEIGHTS

AT ROOF LEVEL

ROOF DL = (0.131 + 0.010) KSF × 50' × 194' = 1368^K

EXT. WALLS @ 4 PSF

N & S $2 \times 194' \times \left(\frac{12'}{2} + 1'\right) \times 0.004$ = 11^K (PARAPET)

E & W $2 \times 50' \times 7 \times 0.004$ = 3

$W_R = 1382^K$

AT FLOOR LEVEL

FLOOR DL = $0.164 \times 50' \times 194'$ = 1591^K

EXT. WALLS

N & S $(12'/7') \times 11$ = 19

E & W $(12'/7') \times 3$ = 5

$W_3 = W_2 = 1615^K$

TOTAL W = $\Sigma W = 1382 + 1615 + 1615 = 4612^K$

BASE SHEAR Para. 3-3(D)

$Z = 1.0$ $I = 1.0$ $K = 0.67$

$T = 0.10N = 0.10(3) = 0.30$ $C = \frac{1}{15\sqrt{T}} = 0.12$

T_s IS UNKNOWN ∴ USE $S = 1.5$

$CS = 0.12 \times 1.5 = 0.18$, BUT NEED NOT EXCEED 0.14

$V = ZIK(CS)W = 1.0 \times 1.0 \times 0.67 \times 0.14 \times W = 0.0938W$

$= 0.0938 \times 4612 = 432^K$

STORY FORCES & OVERTURNING

LEVEL	h_x	Δh	w_x	$w_x h_x$	$\frac{wh}{\Sigma wh}$	F_x	V_x	$V_x h$	M
R	36'		1382	49,752	.46	200^k			
		12'					200	2400	
3	24'		1615	38,760	.36	156^k			2400
		12'					356	4272	
2	12'		1615	19,380	.18	76^k			6672
		12'					432	5184	
			4612	107,892	1.00	432^k			11,856^k
			W			V			

RELATIVE RIGIDITIES OF FRAMES

THE FOLLOWING ASSUMPTIONS ARE MADE IN ORDER TO ESTIMATE THE FORCES TO BE APPLIED IN THE FRAME ANALYSIS.

LONGITUDINAL FRAMES

A, B & C   5 COLUMNS @ 1 = 5.0
2 COLUMNS @ 3/4 = 1.5   R = 6.5

$\Sigma R = 3 \times 6.5 = \underline{19.5}$

TRANSVERSE FRAMES

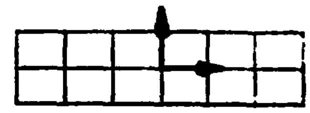
LINES 1 & 7   1 COLUMN @ 1 + 2 @ 3/4 = 2.5
ADJUST FOR SHORTER BEAMS = 32'/24' = 1.33;
SAY 1.16* x 2.5 = 2.9
ADJUST FOR NARROW BEAMS = (26"/28") = 0.928;
SAY 0.94* x 2.9 = $\underline{2.7}$

LINES 2-6   1 COLUMN @ 1 + 2 @ 3/4 = 2.5
ADJUST FOR SHORTER BEAMS   1.16 x 2.5 = 2.9
ADJUST FOR SHALLOWER BEAMS = $(24/30)^3$ = 0.51;
SAY 0.75* x 2.9 = $\underline{2.2}$

$\Sigma R = (2 \times 2.7) + (5 \times 2.2) = \underline{16.4}$

* NOTE: Effects of joint rotation are not proportional to beam stiffness.

DISTRIBUTION OF FORCES TO FRAMES



UNIT FORCE, $F = 1.00^K$

FRAME	REL R	$\frac{R}{\Sigma R}$	DIRECT FORCE	d	d^2	Rd^2	$\frac{Rd^2}{\Sigma Rd^2}$	TORSION FORCE	DIRECT + TORSION
1	2.7	.165	.165	+96	9216	24,883	.312	+.031	.196 •
2	2.2	.134	.134	+64	4096	9,011	.113	+.017	.151 •
3	2.2	.134	.134	+32	1024	2,253	.028	+.008	.142 •
4	2.2	.134	.134	0	0	0	0		.134 •
5	2.2	.134	.134	-32	1024	2,253	.028	-.008	.126
6	2.2	.134	.134	-64	4096	9,011	.113	-.017	.117
7	2.7	.165	.165	-96	9216	24,883	.312	-.031	.134
	16.4	1.000	1.000 TRANSV.						
A	6.5	.333	.333	+24	576	3,744	.047	+.019	.352 •
* B	6.5	.334	.334	0	0	0	0	0	.334 •
C	6.5	.333	.333	-24	576	3,744	.047	-.019	.352
	19.5	1.000	1.000 LONGIT.		$\Sigma Rd^2 =$	79,782			

• = DESIGN ALL FRAMES FOR POSITIVE TORSION

TORSION:

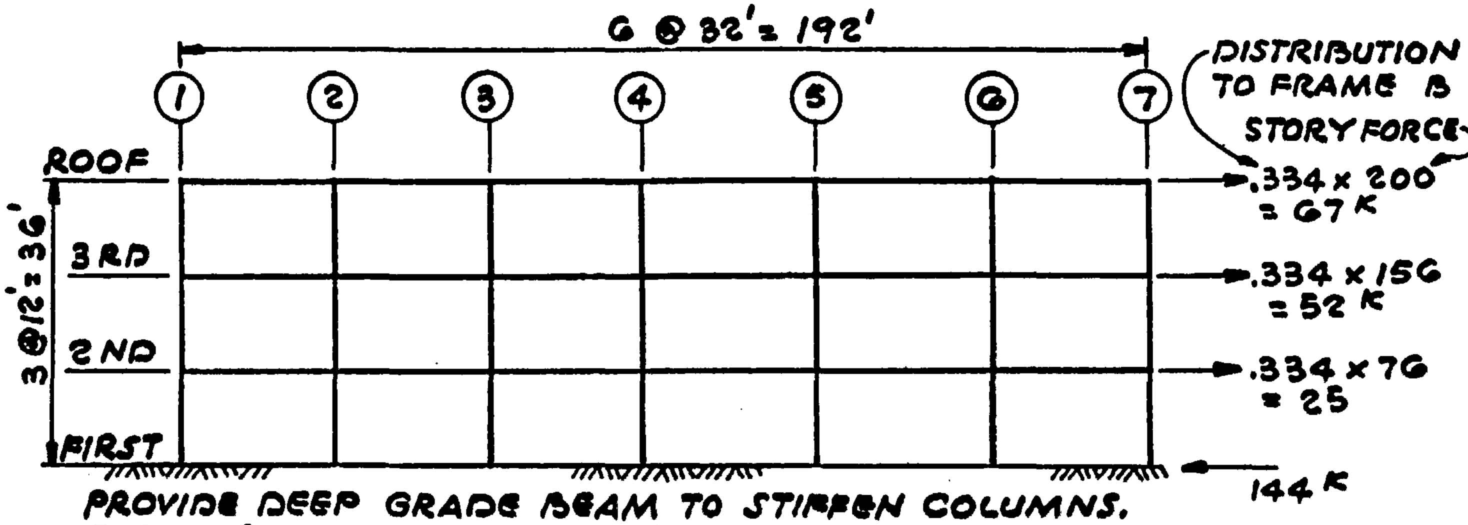
BUILDING IS SYMMETRICAL, ∴ NO CALCULATED TORSION. ACCIDENTAL TORSION IS BASED ON ECCENTRICITY OF 5% OF MAX. DIM.

$$M_T = [0.05(192')] \times F = 9.6' \times 1.00 = 9.6\ K'$$

FORCE TO FRAMES

$$= \frac{(Rd^2/\Sigma Rd^2)\, M_T}{d} = \frac{9.6}{d} \cdot \frac{Rd^2}{\Sigma Rd^2}$$

* IN DESIGN EXAMPLE, FRAME B TAKES $0.334 \times F_x$



PROVIDE DEEP GRADE BEAM TO STIFFEN COLUMNS.
EXAMPLE ASSUMES COLUMNS FIXED AT BASE.

BEAM	b x D	I_g ft⁴	W_{DL}	W_{LL}	REDUCED LL
ROOF	28"x30"	3.04	3.14 K/'	0.27 K/'	12 PSF
3RD & 2ND	28"x30"	3.04	3.94 K/'	0.72 K/'	30 PSF

ALL COLUMNS 24"x24"
$I_c = 1.33$ FT^4

$f' = 4000$ PSI
(STONE AGG.)

$E = 57{,}000 \times \sqrt{4000}$
$= 3{,}605{,}000$ PSI
$= 519{,}000$ KSF

<u>REQUIRED OUTPUT</u>

- BEAM & COLUMN END MOMENTS AT FACE OF SUPPORT
- COLUMN SHEAR & AXIAL LOAD
- FRAME PERIOD AND HORIZONTAL DEFLECTION UNDER LATERAL LOADS

<u>BEAM DIMENSION</u>

<u>LIMITATIONS</u> (FIG 7.2)

WIDTH = 28" > 10" MIN
DEPTH = 30 < 3.33 b MAX.
WIDTH = 28 < COL. WIDTH + 1.5d MAX

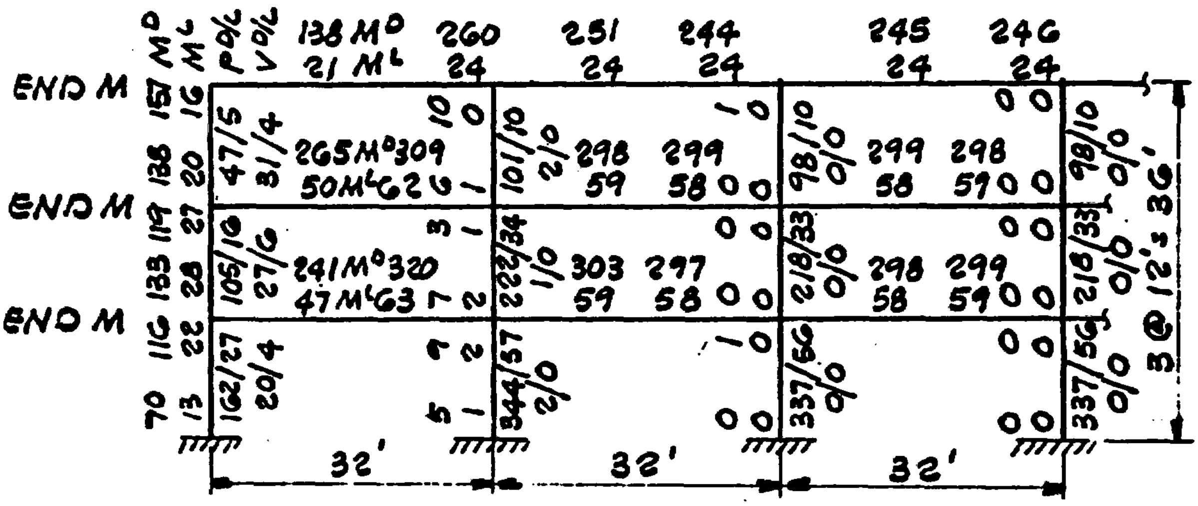
<u>FRAME ANALYSIS</u> = INPUT - FRAME B

FRAME ANALYSIS RESULTS (calcs not shown) FRAME B

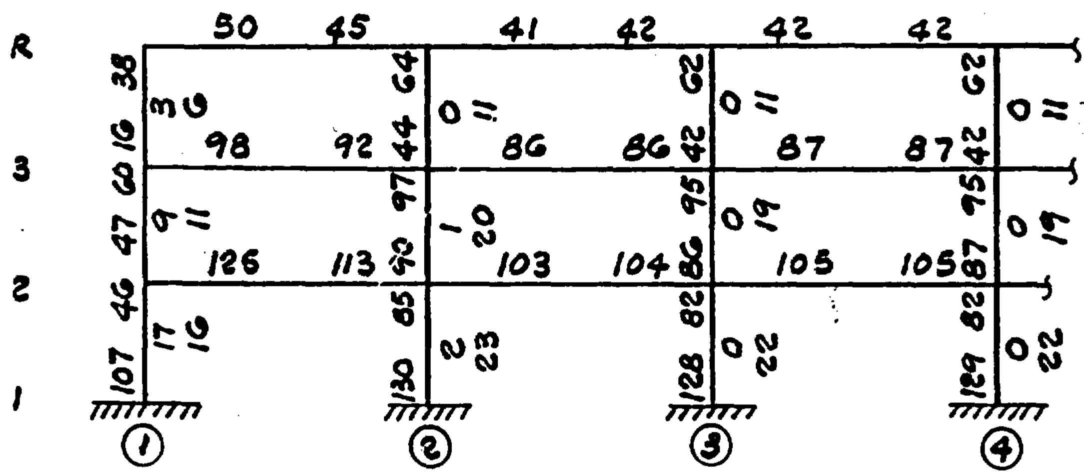
	DL	LL	SEISMIC F
ROOF	3.14	0.27 k/'	67 k
3RD	3.94	0.72	52
2ND	3.94	0.72	25

Values of moments & shears are taken at face of column or beam.

ALL BEAMS 28"x 30"
ALL COLUMNS 24"x24"



VERTICAL LOAD



LATERAL LOAD

APPROX. 2ND STORY DRIFT.

$\Delta_{col} = \frac{Ph^3}{12E_cI_c} = \frac{20(12)^3}{12(3600)1.33(12)} = .05"$ $\quad \Delta_{TOTAL} = (.05+.055)1/.67$

$= .157"$ O.K.

$\Delta_{GIR} = \frac{Plh^2}{12E_cI_b} = \frac{20(30)(12)^2}{12(3600)3.04(12)} = .055"$ $\quad \Delta_{ALLOW.} = .005h = .72"$

DESIGN FORCES FOR BEAMS - PROCEDURE

1. Obtain end M's and V's at face of support. These are given on p. 10 for Frame B.

2. Calculate and tabulate factored M's and V's.

   a. Vertical load only

      $1.4D + 1.7L$

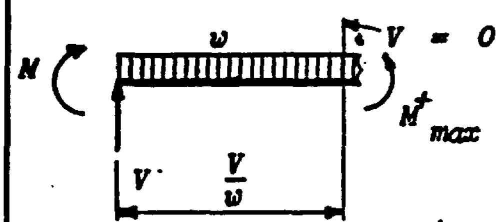
   b. Vertical plus maximum increase due to seismic

      $1.4\ (D+L+E)$ when E is in direction adding to $-M$

   c. Vertical minus reverse loading due to seismic

      $0.9D + 1.4E$ when E is in direction giving $+M$

3. Calculate and tabulate max. pos. mom. away from the end of the beam:



$+M$ produces tension on the bottom

$+V$ is upward at the left end of the beam

$$M^{+}_{max} = M + \frac{V^2}{2w} \quad (wL > V > 0)$$

4. Select maximum values for design. It is strongly recommended to sketch moment diagrams, especially when spans and loads are irregular.

5. Checkerboard loading may govern, maximum positive moments.

DESIGN FORCES FOR COLUMNS

1. Obtain P, M, V at face of support. These are given on p. 10 for Frame B

2. Calculate and tabulate factored M's and P's

   a. $1.4\ D$
   b. $1.4D + 1.7L$
   c. $1.4(D+L+E)$ for E in direction adding to vert. load
   d. $0.9D+1.4E$ for E in direction opp. to vert. load

BEAM FORCES

FRAME (B) FLOOR 2 FROM (1) TO (2)

28"×30" d = 27½ +M = TENS. ON BOTTOM	END 1		CLEAR SPAN = 30.0'			END 2	
	M	V	w	wL'	M+	M	V
(P.10) D	−241 K'	+56.5	3.94 K/'	119 K	164	−820	+62
L	−47	+10.6	0.72	22	31	−63	+12
E ⇄	±125	∓8	−	−	SMALL	∓113	±8
1.4D + 1.7L	−417	+97				−555	+106
M+					+282		
$1.4(D+\overrightarrow{L+E})$ M+	−228	+88			+273	−694	+114
$1.4(D+\overleftarrow{L+E})$ +M	−578	+105			+273	−378	+92
$0.9D+\overrightarrow{1.4E}$	−40	+40				−446	+67
$0.9D+\overleftarrow{1.4E}$	−392	+62				−130	+44.5
MAX. NEG.	−578				−	−694	
MAX. POS.	−*				+282	−*	
1.4(D+L)		94					103
1.1(D+L)		74					81

* IN THIS EXAMPLE, THE SEISMIC MOMENTS ARE NOT LARGE ENOUGH TO CAUSE LOAD REVERSAL.

BEAM LONGITUDINAL REINFORCEMENT

$f_y = 60$
$f'_c = 4$
$m = \frac{f_y}{0.85 f'_c} = 17.65$

① FRAME B FLOOR 2 ②

$w_D = 3.94^{K}/_{l}$
$w_L = 0.72^{K}/_{l}$

28" x 30" d = 27.5" F = 1.76

$l' = 30.0'$

	①		②	
−M	−578		−694	−651
$K^* = \frac{M}{F}$	328		394	
a_u	4.24		4.19	
REQ'D $A_s = \frac{M}{a_u d}$	4.96		6.02	
TOP BARS	5-#9		6-#9	
ACTUAL A_s	5.00		6.00	MIN. ρ = .0033
ρ	.00649		.00779	MAX. ρ = .025
+M		308		
K^*		175		
a_u		4.37		
REQ'D A_s		2.56		
½ TOP A_s	2.48		3.01	
BOTT. BARS	3-#9	4-#9	3-#9	
ACTUAL A_s	3.00	4.00	3.00	
ρ	.00390	.00519	.00390	

ULTIMATE MOMENT CAPACITY FURNISHED M_u

K^*	330		391	
$-M_u = KF$	581		688	
K^*	203	269	203	
$+M_u$	357	478	357	

ULT. MOM. CAP'Y: φ = 1.0 & STEEL AT 1.25 f_y ** M_p

K^*	406 ÷ 0.9 = 451		481 ÷ 0.9 = 534	
$-M_p$	794		940	
K^*	250 ÷ 0.9 = 278		250 ÷ 0.9 = 278	
$+M_p$	489	641	489	

*ACI SP17 VOL. 1, FLEX 1.2

** Solve by modifying ρ by 1.25 factor.

BEAM TRANSVERSE REINFORCEMENT

(1) FRAME B FLOOR 2 (2)

$\omega_D = 3.94$
$\omega_L = 0.72$

28" x 30" $d = 27.5"$
$\ell' = 30.0$

$f_y = 60$
$f'_c = 4$
$\phi = 0.85$

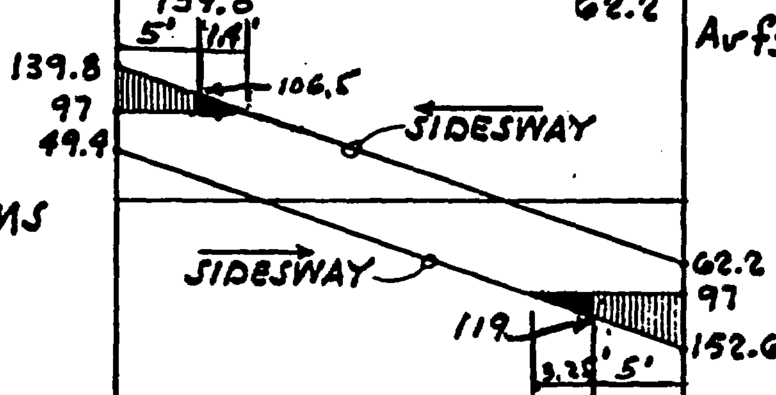
$$V_u = \frac{M_p^1 + M_p^2}{L} + 1.4\,V_{D+L}$$

$v_c = 2\sqrt{f'_c} = 126$
$v_u = 8\sqrt{f'_c} = 504$ MAX.
$V_c = .126 \times 28 \times 27.5 = 97^K$

	(1)	(2)
$\overrightarrow{M_p}$	+489	−940
$\Sigma M/L$	−47.6	+47.6
$1.4\,V_{D+L}$	97	105
V_u	49.4	152.6
$\overleftarrow{M_p}$	−794	+489
$\Sigma M/L$	+42.8	−42.8
V_u	139.8	62.2

#3 (stirrup)
$A_v f_y = 3 \times .11 \times 60 = 19.8^K$

SHEAR DIAGRAMS



	(1)	(2)	
$v_u = \dfrac{V}{\phi b d^2}$	$\dfrac{139{,}800}{0.85 \times 28 \times 27.5} = 214$	233	MIN. STIRRUPS
$v_u - v_c$	88	107	$\dfrac{d_1}{4} = 6.9"$ ←
$s = \dfrac{A_v f_y}{(v_u - v_c)\,b}$	8.03"	6.61"	8 BAR DIA = 8"
MAX. SPACING (WITHIN 2D = 5'-0" FROM COL.)	7"	6"	24 TIE DIA. = 9"

MAX $S = \frac{d}{2} = 13.5"$ → $\boxed{\text{use 13" spacing}}$

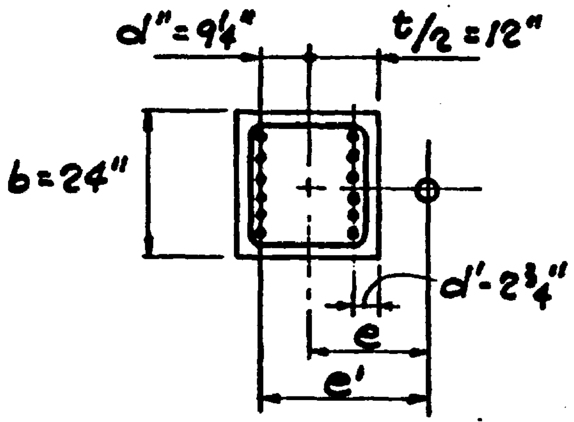
BEYOND 5'-0" (WITHIN SHADED AREA):
$v_u - v_c$: 36 (S = 19") (S = 12.6") 56

REF. FIG. 7-6

COLUMN FORCES FRAME (B)

1ST STORY	COLUMN B-1			COLUMN B-2		
	AXIAL	MOMENT		AXIAL	MOMENT	
		TOP	BOTTOM		TOP	BOTTOM
D	162	116	70	344	-9	-5
L	27	22	13	57	-2	-1
E →	-17	-46	-107	-2	-85	-130
E ←	+17	+46	+107	+2	+85	+130
1.4D + 1.7L	272.7	199.8	120.1	578.8	-16.0	-8.7
$1.4(D+L+\overrightarrow{E})$	240.8	128.8	-33.6	558.6	-134.4	-190.4
$1.4(D+L+\overleftarrow{E})$	288.4	257.6	266.0	564.2	103.6	173.6
$0.9D+1.4\overrightarrow{E}$	122.0	40.0	-86.8	306.8	-127.1	-186.5
$0.9D+1.4\overleftarrow{E}$	169.6	168.8	212.8	312.4	110.9	177.5

COLUMN PROPERTIES (B-2)



$f'_c = 4000$ PSI
$f_y = 60{,}000$ PSI
6-#10 EACH FACE
$A_s = A'_s = 7.62\ IN^2$
$d = 21\frac{1}{4}''$
$d''/d = 0.129$
$\phi = \frac{18\frac{1}{2}''}{24''} = 0.77$

DIMENSIONAL LIMITATIONS:
WIDTH = 24" > 12" OK
$\frac{MIN.DIM.}{MAX.DIM.} = \frac{24}{24} = 1 > 0.4$ OK

$\left.\begin{array}{l} E_c = 519{,}000\ KSF \\ I_c = 1.33\ FT^4 \end{array}\right\}$ P. 9
$E_c I_c = 690{,}000\ K \cdot FT^2$
$r = 0.3t = 0.60\ FT$
$L_u = 9.5\ FT$

COLUMN SLENDERNESS FRAME (B)

1ST STORY	COLUMN B-1 TOP	COLUMN B-1 BOTTOM	COLUMN B-2 TOP	COLUMN B-2 BOTTOM
$K = I/L$	$1.33/9.5 = 0.14$			
ΣK (COLS)	0.28	0.14	0.28	0.14
BEAM I/L	$3.04/30 = 0.10$			
ΣK (BMS.)	0.10	∞	0.20	∞
$\psi = \frac{\Sigma K\ COL}{\Sigma K\ BM}$	2.8	1 FIXED END	1.4	1 FIXED END
k	1.54		1.37	
kL/r	$1.54 \times 9.5/0.6 = 24.4$ > 22 ∴ "SLENDER" (CONT. BELOW)		$1.37 \times 9.5/0.6 = 21.7$ < 22 ∴ "SHORT" (CONT. ON P.17)	

	MAX. AXIAL	MIN. AXIAL	REMARKS
P_u	288.4	122.0	Σ AXIAL, FRAME B
ΣP_u (ALL COLS)	3440 ←		$(2 \times 272.7) + (5 \times 578.8)$
$\beta_d = \frac{M_D}{M_T}$	$\frac{116}{258} = 0.450$	$\frac{116}{40} = 2.90$	$EI = \frac{\frac{E_c I_c}{2.5}}{1 + \beta_d}$
EI	190,000	70,800	
$P_c = \frac{\pi^2 EI}{(KL_u)^2}$	8760	3260	
ΣP_c (ALL COLS)	72,900		$(2 \times 8760) + \left(5 \times \left(\frac{1.54}{1.37}\right)^2 \times 8760\right)$
$\delta = \frac{C_m}{1 - \frac{P_u}{\phi P_c}}$	1.049	1.056	$C_m = 1.0$ FOR UNBRACED COLUMNS $\phi = 0.7$
$\delta = \frac{C_m}{1 - \frac{\Sigma P_u}{\phi \Sigma P_c}}$	1.072		MAX. COL. $M_u = 266$ K' (P.15)
δM	$1.072 \times 266.0 = 285$ K'	$\approx 1.07 \times 86.8 = 93$ K'	REQ'D DES. M_u (NOTE: OTHER P.15 COMBIN. OF P & M WERE ALSO INVESTIGATED)

COLUMN CAPACITY — FRAME (B)

1ST STORY	COLUMN B-1	COLUMN B-2
SIZE	24" x 24"	24" x 24"
BARS	8 - #9 $A_{st} = 8.00$	12 - #10 $A_{st} = 15.24$
$\frac{P_u}{A_g} = \frac{\phi P_n}{A_g}$	$\frac{288.4}{24 \times 24} = 0.500$ KSI	$\frac{558.6}{24 \times 24} = 0.970$

COL. MOM. CAPACITY MUST BE GREATER THAN BEAM CAPACITY SINCE $P/A_g > 0.12 f_c' = 0.48$ KSI

$\rho_g = \frac{A_{st}}{A_g}$	$\frac{8.00}{24 \times 24} = 0.0139$	$\frac{15.24}{24 \times 24} = 0.0264$

USING ACI SP17A-78 CHART "COL. E4 - 60.75" FIND

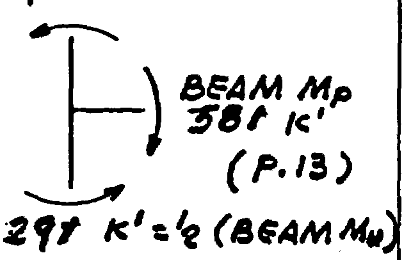
M_u	485K' $> (M_u = 285)$	785 K' $> M_u = 190.4$)

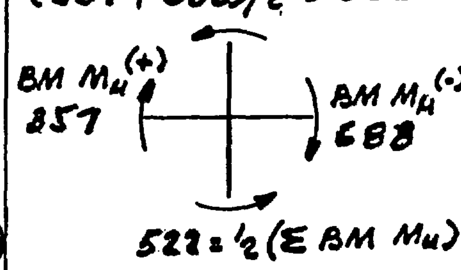
COL. MOM CAPACITY, M_p @ $\phi = 1$, STEEL @ $1.25 f_y$
(which may be approximated by using $1.25 A_s$)

Chart E4-60.75	$\rho = 1.25 \times 0.0139 = 0.174$	$\rho = 1.25 \times 0.0264 = .033$
	$\frac{\phi M_n}{A_g h} = 0.48$	$\frac{\phi M_n}{A_g h} = 0.79$
	$M_p = \frac{0.48 \times 24^3}{0.7 \times 12} = 790^{k'}$	$M_p = \frac{0.79 (24)^3}{0.7 \times 12} = 1300^{k'}$

CHECK COL. CAP'Y > BEAM CAP'Y @ $\varphi = 1$, STEEL @ $1.25 f_y$

	COLUMN B-1	COLUMN B-2
	COL. M_u SHOULD EXCEED 581/2 = 291 K'	COL. M_u SHOULD EXCEED (357 + 688)/2 = 522

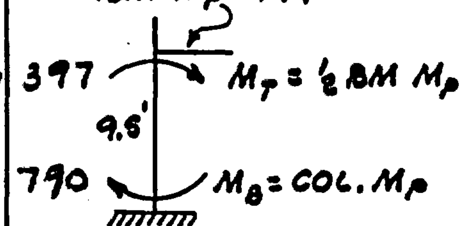
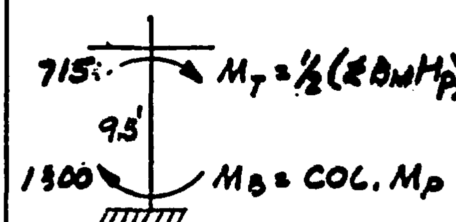


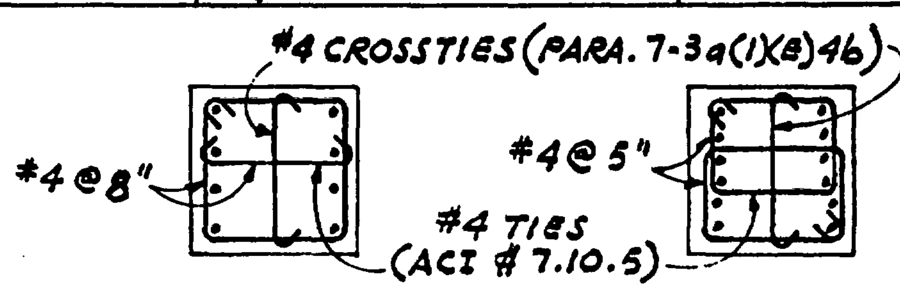


COL. M_u = 485 > 291 OK

COL. M_u = 785 > 522 OK

COLUMN SHEAR — FRAME B

1ST STORY	COLUMN B-1	COLUMN B-2
REFER TO FORMULA 7-7, PARAGRAPH 7-3a (1)(E)5.	BM YIELDS BEFORE COL BM M_p = 794 	
V	$\frac{397+790}{9.5'} = 125$ K	$\frac{715+1300}{9.5'} = 212$ K
$v_u = \frac{V}{\varphi A_c}$	$\frac{125}{.850 \times 420} = 0.35$ KSI (420 ← P. 19)	$\frac{212}{.85 \times 420} = 0.59$
$v_u - v_c$	.350 − .126 = .224 KSI	.59 − .126 = .454 KSI
TIE $S = \frac{A_v f_y}{(v_u - v_c) d_c}$	$\frac{3 \times 0.20^{\square''} \times 60^K}{.224 \text{ KSI} \times 20.5''} = 7.81''$ USE 8"	$\frac{4 \times 0.20^{\square''} \times 60^K}{.454 \text{ KSI} \times 20.5''} = 5.16$ USE 5"



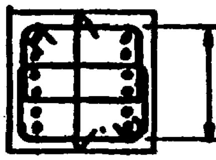
SINCE $P/A > 0.12 f'_c$, $V_c = 2\sqrt{f'_c} = 126$ PSI = 0.126 KSI

USE #4 COLUMN TIES, $A = 0.20^{\square''}$

MAX. SPACING, $S_{MAX} = \frac{\text{COL. DIM.}}{2} = 12''$

COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT

FOR CONFINEMENT



$h'' = 24 - 2(1\tfrac{3}{4}) = 20.5''$

TIE SETS @ SPACING a, $a \leq 4''$

A''_{sh} = TOTAL AREA OF HOOP

$A_g = 24 \times 24 = 576 \text{ IN}^2$

$A_c = 20.5 \times 20.5 = 420 \text{ IN}^2$

$f'_c = 4{,}000 \quad f''_{yh} = 60{,}000$

REF. PARA. 7-3a(1)(E)4

① $$a = \frac{A''_{sh}}{0.30h'' \frac{f'_c}{f''_{yh}} \left(\frac{A_g}{A_c} - 1\right)} = \frac{A''_{sh}}{0.30 \times 20.5 \times \frac{4}{60}\left(\frac{576}{420} - 1\right)} = 6.57 A''_{sh}$$

LARGER GOVERNS

② $$a = \frac{A''_{sh}}{0.12h'' \frac{f'_c}{f''_{yh}}} = \frac{A''_{sh}}{0.12 \times 20.5 \times \frac{4}{60}} = 6.10 A''_{sh}$$

BAR SIZE	A_s	A''_{sh}	$6.57A''_{sh}$
#3	0.11	0.44	2.89
#4	0.20	0.80	5.26 ← USE #4 GR 60 @ 4"
#5	0.31	1.24	8.15

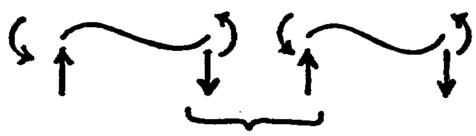
EXTENT OF SPECIAL TRANSV. REINF. IS THE MAXIMUM OF:
- MAX. COL. DIMENSION = 24" ←
- 1/6 CLEAR HEIGHT = 114/6 = 19"
- 18"

EXTEND MIN. 2'-0" ABOVE & BELOW

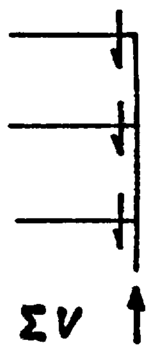
CONTINUED →

COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT-CONT'D

SPECIAL TRANSV. REINF. IS ALSO REQUIRED WHERE COLUMN CAPACITY IS LESS THAN THE SUM OF THE SHEARS ABOVE. REF. PARA. 7-3a(1)(8) 4d.



- ΣV IS THE COLUMN LOAD AT THIS LEVEL.
- AT INTERIOR COL'S THIS SUM IS RELATIVELY SMALL.
- END COLUMNS ARE USUALLY CRITICAL.



ΣV

1. INCLUDE ALL BEAMS ABOVE THE COLUMN IN QUESTION.

2. $V_{u_i} = \dfrac{(M_p^A + M_p^B)}{L}\Bigg|_{p.13} + 1.1\,V_{D+L}\Bigg|_{p.12}$

3. AT THE COLUMN IN QUESTION, CALCULATE THE MAX. MOMENT TRANSFERRED TO THE COLUMN BY THE YIELDING BEAM.

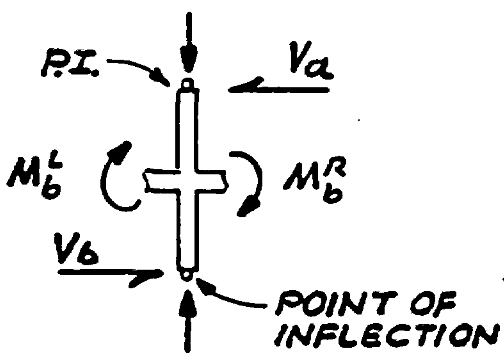
4. DOES THE COLUMN HAVE THE CAPACITY TO CARRY ΣV_u WITH THIS BEAM MOMENT?
   YES: NO ADD'L REINF. REQ'D.
   NO: PROVIDE THE SPECIAL TRANSV. REINF. CALCULATED ON P. 19 FOR FULL HEIGHT.

SEE P. 21 FOR SAMPLE CALC.

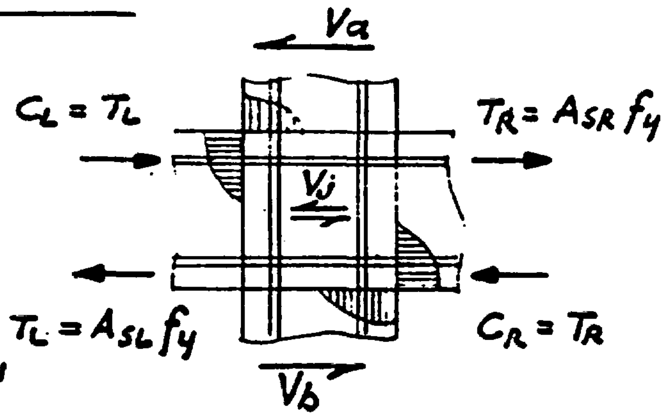
COLUMNS: SPECIAL TRANSVERSE REINFORCEMENT CONT.

FRAME COLUMN	B 1	
ROOF BEAM $\Sigma M_p/L$ $1.1\ V_{D+L}$ V_u^R	35 61 96	CALCS. NOT SHOWN
3RD FLR. BEAM $\Sigma M_p/L$ $1.1\ V_{D+L}$ V_u^3	43 76 119	ASSUME SAME AS 2ND
2ND FLR. BEAM $\Sigma M_p/L$ $1.1\ V_{D+L}$ V_u^2	43 76 119	$= \frac{794 + 489}{30}$ (p. 13) (p. 12)
ΣV_u ABOVE	334	
M_p FROM BM	397	$= \frac{1}{2}$ BM $\quad M_p = \frac{794}{2}$
ALLOW COL. M WITH $P = \Sigma V_u$	659	(SP 17A VOL. 2 CHART 64-60-75)
COL. M > M_p	YES	
SPEC. TRANSV. REINF.	NO	

BEAM - COLUMN JOINT



FORCES ON COLUMN



FORCES ON BEAM COLUMN JOINT

THE JOINT SHEAR, $V_j = A_{SR} f_y + C_L - V_a$

$$= (A_{SR} + A_{SL}) f_y - V_a$$

$$v_j = V_j / \phi b d$$

TIE SPACING, $S = (A_v f_y)/((v_j - v_c)\, b)$

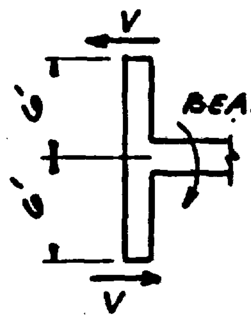
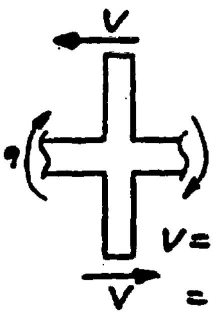
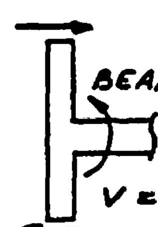
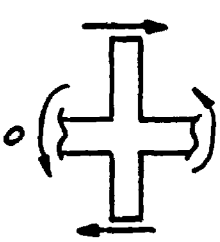
NOTE: $v_c = 0$ IF $P_u/A_g < 0.12 f'_c$

TIE REINFORCEMENT: USE THE MIN. SPAC'G FROM

1. SPECIAL TRANSVERSE REINFORCEMENT (p. 19)
2. COLUMN SHEAR (p. 18)
3. JOINT SHEAR (ABOVE) CALCULATED AS FOR COL SHEAR

- WHERE BEAMS FRAME INTO ALL FOUR SIDES OF COLUMNS, USE ONE HALF OF THE REINFORCING CACULATED ABOVE.
- WHERE CORNER OF A TIED COLUMN EXTENDS 4" OR MORE BEYOND CONFINING BEAMS, THE FULL TRANSVERSE REINFORCEMENT SHALL BE PROVIDED THROUGH THE CONNECTION AND AROUND BARS OUTSIDE OF THE CONNECTION.
- WHERE LONGITUDINAL REINFORCEMENT OUTSIDE THE CORE IS UNCONFINED BY ANOTHER BEAM, PROVIDE THE FULL REINFORCEMENT THROUGH THE CONNECTION.

BEAM — COLUMN JOINT — CONT.   FRAME (B)

2ND STORY	COLUMN B-1	COLUMN B-2
HOOPS IN JOINT ← E	 BEAM $M_p = 794$ P. 13 $V = \frac{794^{K'}}{12'} = 66.2^K$	 489 940 $V = \frac{940+489}{12} = 119^K$
BEAM A_{SR}	$5.0\ IN^2$ (5-#9)	6.0 (6-#9)
BEAM A_{SL}	0	3.0 (3-#9)
$(A_{SR}+A_{SL})F_Y$	$(5.0+0)(60) = 300^K$	$(6.0+3.0)(60) = 540^K$
V_u (see Fig 7-9)	$300-66 = 234^K$	$540-119 = 421^K$
$v_u = \frac{V_j}{\phi bd}$	$\frac{234{,}000}{0.85 \times 24 \times 20.5} = 560\ psi$	$\frac{421{,}000}{0.85 \times 24 \times 20.5} = 1007\ psi$
$\frac{P_u}{A_g}$	$\frac{169{,}600}{24 \times 24} = 294\ psi$ $< 0.12 F_c' = 480$	$\frac{312{,}400}{24 \times 24} = 542\ psi$ $> 0.12 F_c'$
v_c	0	126
→ E	 BEAM $M_p = 489$ P. 18 $V = \frac{489}{12} = 40.8$	 940 489 $V = 119$
BEAM A_{SR}	$3.0\ IN^2$	$3.0\ IN^2$
BEAM A_{SL}	0	$6.0\ IN^2$
V_u (See Fig. 7-9)	180^K	540^K
v_u	430	1007 psi
$\frac{P_u}{A_G}$	$\frac{122{,}000}{24 \times 24} = 212\ psi < 480$	$\frac{306{,}800}{24 \times 24} = 533 > 480$
v_c	0	126

BEAM-COLUMN JOINT-CONT. FRAME (B)

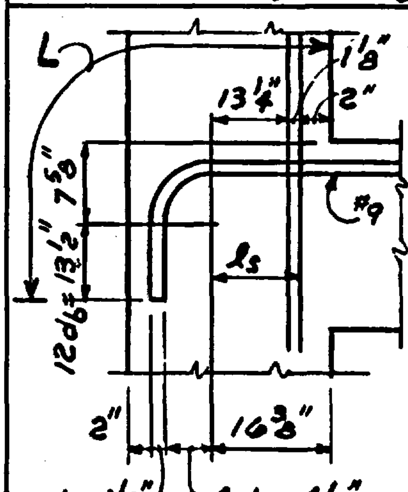
2ND STORY	COLUMN B-1	COLUMN B-2
$(v_u - v_c)_{MAX}$	$560 - 0 = 560'$	$1007 - 126 = 881$ $< 15\sqrt{f'_c} = 949$ OK
$s = \frac{A_v F_y}{(v_u - v_c)b}$	$\frac{0.80 \times 60{,}000}{560 \times 24} = 3.57$	$\frac{0.80 \times 60{,}000}{881 \times 24} = 2.27$ $\times 2 = 4.54$ SINCE BEAMS FRAME INTO ALL 4 SIDES
#4 HOOPS	$A_v = 4 \times 0.20 = 0.80$	
HOOP SPACING	$3\frac{1}{2}''$	$4\frac{1}{2}''$

ANCHORAGE OF LONGIT. BARS

$L\,(REQ'D) = 0.56\,\ell_d = 0.56 \times 38 = 21.3''$ OK

$L\,(PROVIDED) = 2'' + 1\frac{1}{8}'' + 13\frac{1}{2}'' + \frac{\pi}{2}(7.62) + 13\frac{1}{2} = 42 > 21.3$

REFER TO:* "RECOMMENDATIONS FOR DESIGN OF BEAM-COLUMN JOINTS IN MONOLITHIC REINFORCED CONCRETE STRUCTURES" ACI-ASCE COMMITTEE 352 ACI TITLE NO. 73-28 (JULY 76)



$\ell_s = 13\frac{1}{4}'' + 1\frac{1}{8}'' = 14\frac{3}{8}''$
$< 15.4''$ MIN.
BUT CONSIDER CLOSE ENOUGH.
(NOTE: ℓ_s IS NOT A CODE REQ'T, BUT AN ACI-352 RECOMM.)*

ℓ_s (ACI-352)* =
$= 0.04 A_b(\alpha f_y - f_h)/\psi\sqrt{f'_c}$

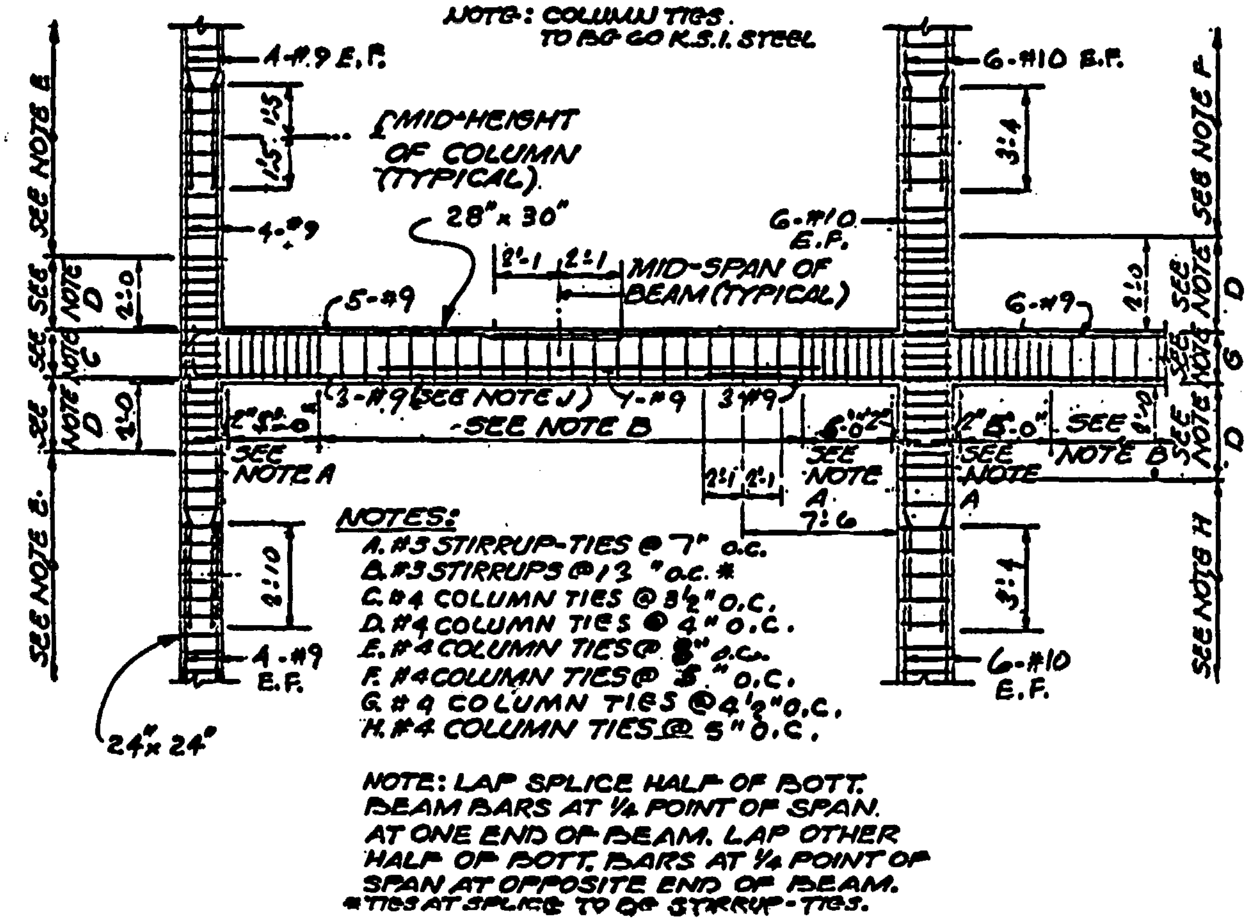
$\alpha = 1.25$

$\psi = 1.4$

$f_h = 700(1 - 0.3 d_b)\psi\sqrt{f'_c}$
$= 700 \times 0.663 \times 1.4 \times 63.2$
$= 41{,}000$ PSI

$\alpha f_y - f_h =$
$= (1.25 \times 60{,}000) - 41{,}000$
$= 34{,}000$ PSI

ℓ_s (MIN) =
$= \frac{0.04 \times 1.0 \times 34{,}000}{1.4 \times 63.2}$
$= 15.4$ IN



LONGITUDINAL FRAME - LINE B

DESIGN EXAMPLE: A-3

BUILDING WITH STEEL MOMENT-RESISTING SPACE FRAMES AND STEEL BRACED FRAMES:

Description of Structure. A three-story Administration Building with transverse ductile moment-resisting frames and longitudinal braced frames in structural steel, using non-bearing, non-shear, exterior walls (skin) of flexible insulated metal panels. There are a series of interior vertical load-carrying column and girder bents in addition to the space frame. The structural concept is illustrated on Sheets 2 and 3.

Construction Outline.

Roof:
- Built-up 5 ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd & 3rd Floors:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Non-bearing, non shear, insulated metal panels.

Partitions:
- Non-structural removable drywall.

Design Concept. The transverse ductile moment-resisting frames are independent of the longitudinal braced frames. The moment frames are designed to K = 0.67; the braced frames to K = 1.00. The metal deck roof system forms a flexible diaphragm; therefore the roof loads are distributed to the frames by tributary area rather than by frame stiffnesses. The metal deck with concrete fill system for the floors form rigid diaphragms and the seismic loads are proportioned to the frames by the frame stiffnesses.

Discussion. Because of the importance of drift of flexible frames, the example shows several stages of design. Preliminary design to find sizes by approximate methods, using different sets of forces for stress and drift. The resulting trial sizes are then used in a computer analysis. (The frames are simple enough to be calculated by hand, but the computer makes short work of calculating design forces, frame period and drift). Final design is discussed, and examples are given for modifications to the results of the computer analysis for accommodating various stress and deflection criteria with consistent sets of member sizes, period, design force, and drift.

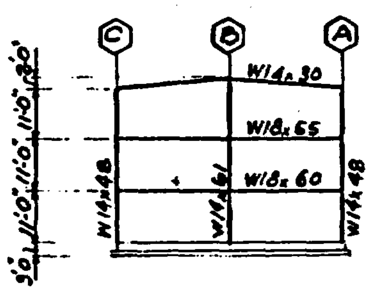
LOADS.

ROOF:

5-PLY ROOFING	=	6.0 P.S.F.
1" INSULATION	=	1.5
STEEL DECK	=	2.3
STEEL PURLINS	=	3.7
STEEL GIRDERS	=	1.2
CEILING	=	10.0
MISCELLANEOUS	=	1.0
DEAD LOAD	=	25.7 P.S.F
ADD FOR SEISMIC: PARTITIONS	=	10.0
TOTAL FOR SEISMIC	=	35.7 P.S.F.

2ND & 3RD FLOORS:

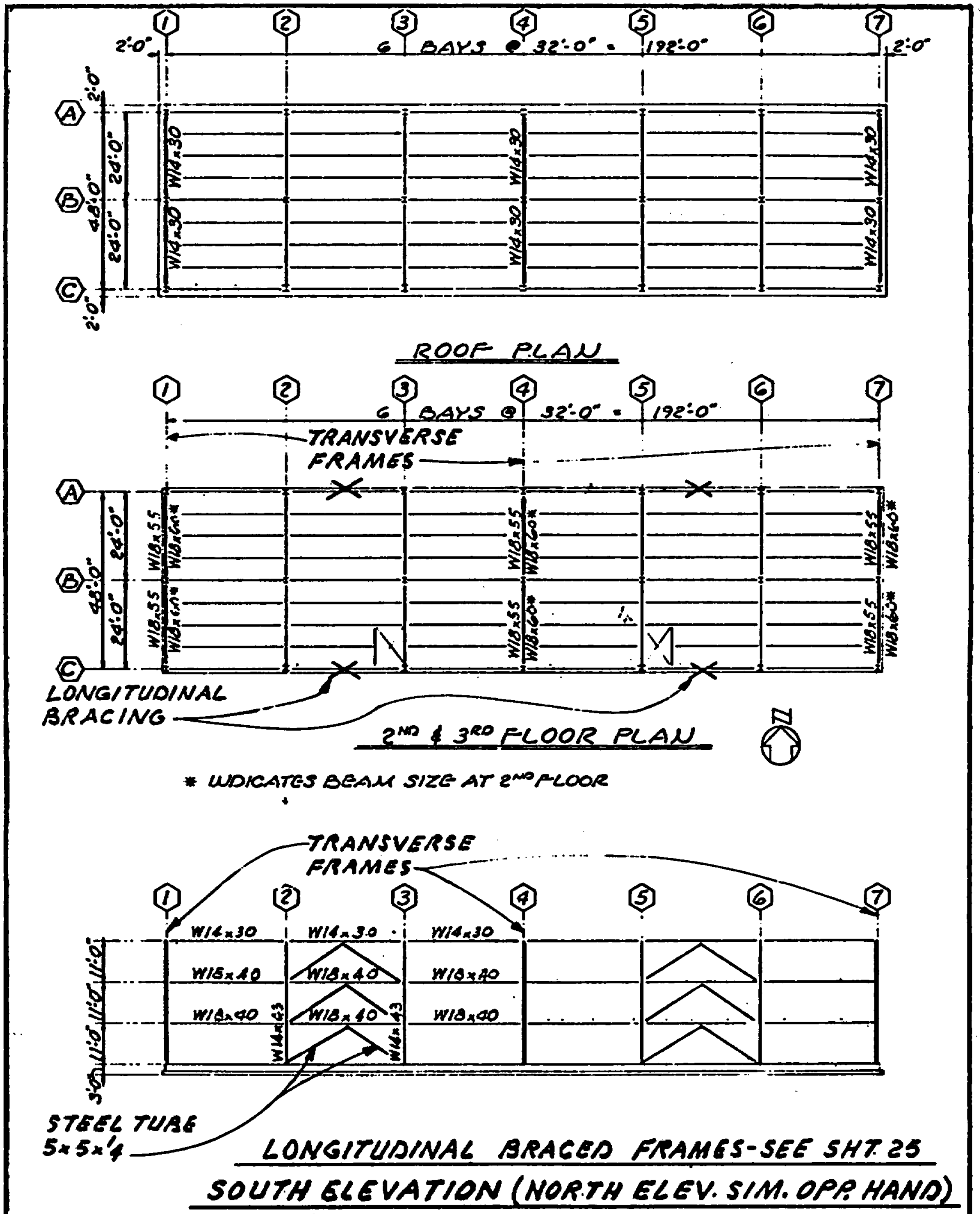
FINISH	=	1.0 P.S.F.
STEEL DECK	=	3.1
CONCRETE FILL	=	32.0
STEEL BEAMS	=	5.9
STEEL GIRDERS & COLUMNS	=	1.6
PARTITIONS	=	20.0
CEILING	=	10.0
MISCELLANEOUS	=	1.0
DEAD LOAD	=	74.5 P.S.F.
LIVE LOAD	=	50.0 P.S.F.



LINES 1, 4, & 7

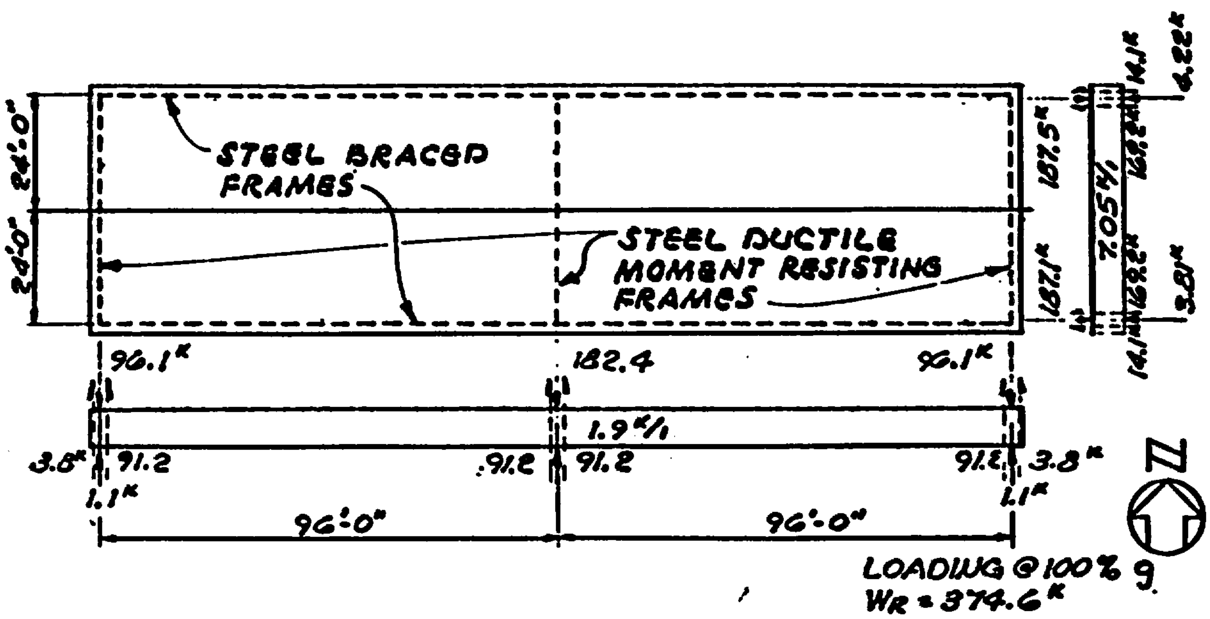
TRANSVERSE DUCTILE MOMENT RESISTING FRAMES

SEE SHT 7



Example A-3 | 3 of 34 | Steel Frames

DESIGN PROCEDURE



LOADS FOR ROOF DIAPHRAGM

EXTERIOR WALLS (NON STRUCTURAL EXTERIOR COVERING)

WALL WT. 5.3 PSF x 5.5' = 29.0 #/1 — FRACTION OF SOLID WALL-WINDOWS OUT

N. WALL = 29 x .75 = 22 x 192' = 4224#

S. WALL = 29 x .68 = 19.8 x 192' = 3801#

41.8#/1

WALL WT. 5.3 PSF x 6' = 31.8 #/1

E. WALL = 31.8 x .76 = 24 x 48' = 1,152#

W. WALL = 31.8 x .76 = 24 x 48' = 1,152#

48#/1

N-S LOADS

ROOF = 35.7 x 52' = 1856.4

WALLS = 41.8 #/1

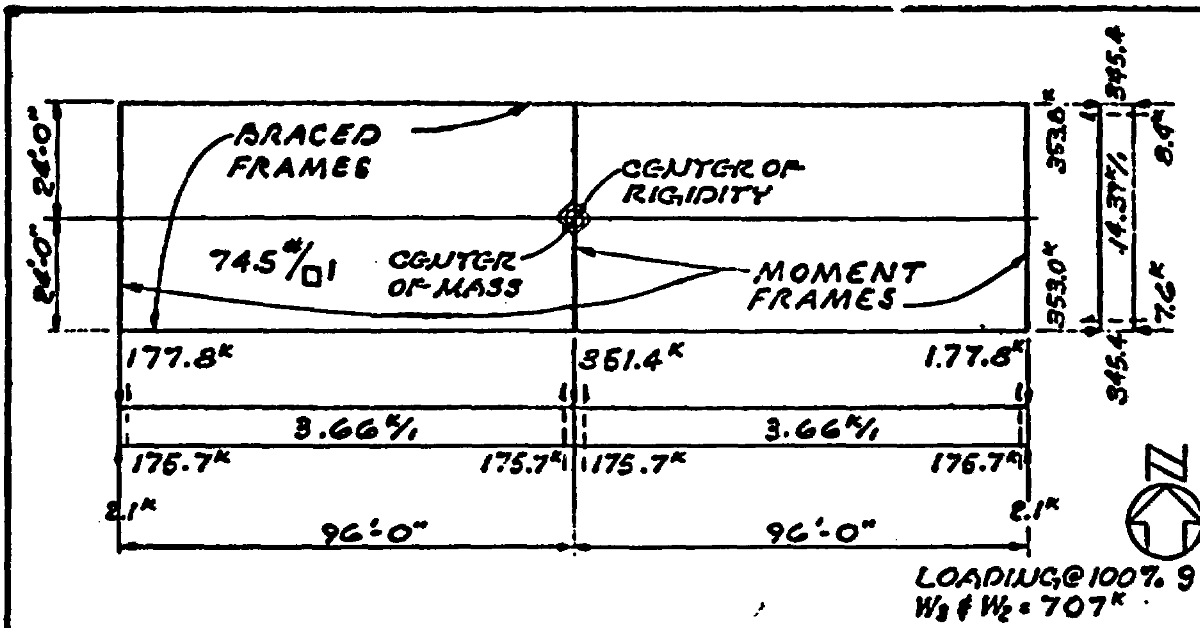
1898.2 #/1

E-W LOADS

ROOF = 35.7 x 196' = 6997.2

WALLS = 48.0 #/1

7045.2 #/1



LOADS FOR 3RD FLOOR DIAPHRAGM (2ND FLOOR SAME)

FLOOR WEIGHT FOR SEISMIC = 74.5 PSF
WALL WT. 5.3 PSF × 11' = 58.3 #/'
N. WALL = 58.3 × .75 = 44 × 192' = 8448#
S. WALL = 58.3 × .70 = 39.6 × 192' = 7603#
88.6 #/'
E. WALL = 58.3 × .75 = 44 × 48' = 2112#
W. WALL = 58.3 × .75 = 44 × 48' = 2112#
88 #/'

N-S LOADS

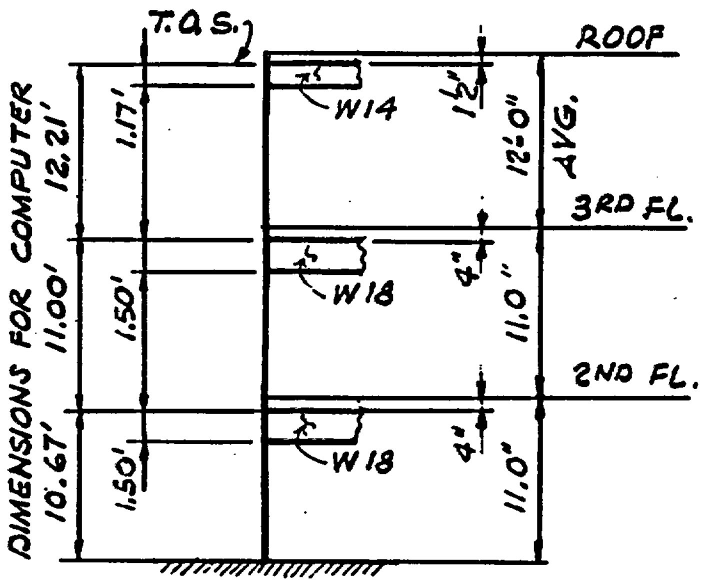
FLOOR = 74.5 × 48' = 3576.0
WALL = 84.0
3660.0 #/'

E-W LOADS

FLOOR = 74.5 × 192' = 14304
WALL 88.
14392. #/'

TRANVERSE (N-S) DIRECTION:

STEEL DUCTILE MOMENT-RESISTING FRAMES



THE COLUMN BASE IS ASSUMED FIXED.

THIS IS NOT ALWAYS FEASIBLE, ACTUAL FOUNDATION CONDITIONS SHOULD BE CAREFULLY STUDIED, AND REALISTIC ASSUMPTIONS SHOULD BE MADE FOR ANALYSIS.

FRAME CHARACTERISTICS

THE MIDDLE FRAME (LINE 4) WILL CARRY HALF THE WEIGHT OF THE BUILDING ACCORDING TO TRIBUTARY AREA. THIS FRAME WILL ALSO TAKE HALF OF THE ROOF LATERAL LOAD BECAUSE THE DIAPHRAGM IS ASSUMED TO BE FLEXIBLE. HOWEVER, ALL THREE FRAMES WILL BE MADE ALIKE SO THAT EACH WILL TAKE A THIRD OF THE FLOOR LATERAL LOADS.

THE DESIGN EXAMPLE WILL CONSIDER THE MIDDLE FRAME AS IT IS MORE HEAVILY LOADED THAN THE END FRAMES.

BUILDING PERIOD

FINDING THE FUNDAMENTAL PERIOD OF A BUILDING WITH ONE OR MORE FLEXIBLE DIAPHRAGMS IS A COMPLEX PROCEDURE.
FOR DETERMINING DESIGN LATERAL FORCES, THE PERIOD WILL BE CALCULATED BY SIMPLE METHODS WHICH ASSUME ALL DIAPHRAGMS RIGID. THIS IS CONSERVATIVE BECAUSE THE EXTRA RIGIDITY RESULTS IN A SLIGHTLY SHORTER PERIOD AND SLIGHTLY LARGER DESIGN FORCES.

BUILDING PERIOD - cont'd

1. $T = 0.1N = 0.3$ sec. (Formula 3-3B)

2. Alternate method: (Chap. 4, Par. 4-3d(3))
Generally, low-rise steel moment frames have T longer than $0.1N$. A more realistic period may be obtained from the following procedure.

   a) Upper Bound: Consider the bare frame governed by drift.*
   The period is approximately $T = 2\Pi\sqrt{\delta_n/a_n}$, where δ_n is the roof deflection in inches, and a_n is the roof acceleration and is approximately $1.7ZICSg$. The formula becomes

   $$T = 0.25\sqrt{\frac{\delta_n}{ZICS}}, \text{ with } \delta_n \text{ in feet and } C = \frac{1}{15\sqrt{T}}$$

   Assuming the average drift for the building is 2/3 the maximum interstory drift, which is limited to 0.005, the roof deflection is $\delta_n = 2/3\ (0.005h_n)$, and the formula above becomes

   $$T = 0.11\left(\frac{h_n}{ZIS}\right)^{2/3}, \text{ with } h_n \text{ in feet.}$$

   and for $Z = 1$, $I = 1.0$, $S = 1.5$, $h_n = 34$; $T = \underline{0.88 \text{ sec}}$
   This will be used for drift.

   b) Limiting Value: Consider the whole building, stiffened by non-seismic frames and non-structural partitions. The period may be estimated as

   $$T = 1.4C_r h_n^{3/4} \quad \text{(Chap. 4, Par. 4-3d(5))}$$

   $$= 1.4 \times 0.035 \times (34)^{3/4} = 0.69 \text{ sec}$$

   This will be used for the initial estimate of forces.

* This assumes that window-wall details are designed to accommodate these deflections. Refer to para. 9-3a and 9-4e.

LATERAL FORCES FOR PRELIMINARY DESIGN

USE $T = 0.69$ SEC. FOR STRESS ANALYSIS.

BUILDING: A-3

DIRECTION: TRANSVERSE

$Z = 1.0;\ I = 1.0\ \ K = 0.67$

$C = 0.08;\ S = 1.5\ \ CS = 0.12$

$W = 1789$ KIPS

$T = 0.69$ SEC.

$F_T = 0.07TV = 0\ V^*$

$F_x = (V - F_T)\dfrac{wh}{\Sigma wh} = 1.0\,V\dfrac{wh}{\Sigma wh}$

$V = ZIKCSW = 0.080\,W$

$= 143$ KIPS

$^*F_T = 0$ WHEN $T \leq 0.7$ SEC.

LEVEL (1)	h FT (2)	Δh FT (3)	w KIPS (4)	Σw (5)	(2)×4 wh (6)	$\frac{wh}{\Sigma wh}$ (7)	F (8)	Σ(7) V KIPS (9)	(3)×(9) ΔOTM K-FT (10)	Σ(10) OTM K-FT (11)	(8)÷(4) $\frac{F}{w}$ (12)	(9)÷(5) $\frac{V}{\Sigma w}$ (13)
						$F_T = 0$						
R	34		375	375	12,750	0.35	50.0				0.133	0.133
		12						50	600			
3	22		707	1082	15,554	0.43	61.5			600	0.087	0.103
		11						111.5	1226			
2	11		707	1789	7,777	0.22	31.5			1826	0.045	0.080
		11						143.0	1573	3399		**
Σ			1787		36,081	1.00	143.0					

** ALL < 0.14, ∴ USE 0.14 FOR DIAPHRAGMS. (CH. 3 PAR. 3(J) 2d)

FOR DRIFT, USE $T = 0.88$ SEC: $C = 1/15\sqrt{0.88} = 0.071$, $S = 1.5$, $CS = 0.107$.
$V = 1 \times 1.0 \times 0.67 \times 0.071 \times W = 0.071\,W = 127^K$ (CH. 4, PAR 5c)

DISTRIBUTION OF FORCES TO FRAMES

Since the roof diaphragm is relatively flexible, the roof forces are distributed by tributary area.

The 2nd and 3rd floor diaphragms distribute the floor forces to the frames according to their relative rigidities.

The transverse frames on lines 1, 4 and 7 are alike, and for preliminary design we may take their rigidity proportional to

see page 9

$$K_1 = \frac{1/3(\text{BASE SHEAR})}{\text{DRIFT}} = \frac{1/3(143)}{2/3(0.005)(34')} = 421 \text{ k/ft}$$

see page 8

The longitudinal frames on lines A and C have a rigidity based on preliminary trials:

see page 25

$$K_A = \frac{1/2(\text{BASE SHEAR})}{\text{DRIFT}} = \frac{1/2(250)}{0.28''/12} = 5360 \text{ k/ft}$$

prelim calcs (not shown)

Use Rel. $K_1 = 1$ and Rel. $K_A = \frac{5360}{421} = 12.7$, say 13

Because of symmetry there is no "calculated" torsion. The "accidental" torsion is the story force, F, times the nominal eccentricity of 5% of the max. building dimension:

$$M_t = F_x \times 0.05 \times 192' = 9.6F_x$$

$$\text{Torsional Shear} = \frac{Kd}{\Sigma Kd^2}\, 9.6F_x$$

DISTRIBUTION OF FORCES - CONT.

FRAME	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR	DESIGN SHEAR
1	1	96	96	9216	$.33 F_T$	$\pm .03 F_T$	$.36 F_T$
4	1	0	0		$.33 F_T$	0	$.33 F_T$
7	1	96	96	9216	$.33 F_T$	$\mp .03 F_T$	$.36 F_T$
			192				
A	13	24	312	7488	$.50 F_L$	$\pm .09 F_L$	$.59 F_L$*
C	13	24	312	7488	$.50 F_L$	$\mp .09 F_L$	$.59 F_L$
			624				
				$\Sigma = 33{,}408$			

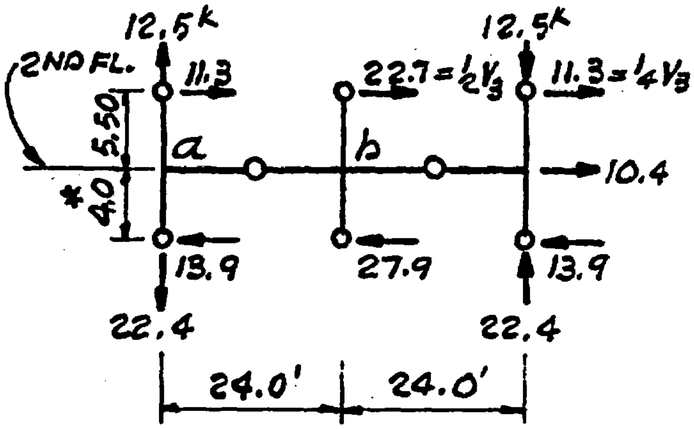
*THESE WILL BE USED FOR DESIGN OF THE LONGITUDINAL FRAMES. SEE P. 25.

DISTRIBUTION TO TRANSVERSE FRAMES

FRAME	1	4	7
ROOF (BY TIBUTARY AREA)			
50.0	x.25 = 12.5	x.50 = 25.0	x.25 = 12.5
3RD. (BY REL. RIGIDITY)			
61.5	x.36 = 22.1	x.33 = 20.3	x.36 = 22.1
2ND.			
31.5	x.36 = 11.3	x.33 = 10.4	x.36 = 11.3
143.0^K	45.9^K	55.7^k	45.9^k

PRELIMINARY DESIGN

MEMBER FORCES BY PORTAL METHOD - FRAME 4



EXTERIOR COLUMN, (MOM. AT ℄ GIRD.)

ABOVE a, $M = 11.3^k \times 5.50' = 62.2$

BELOW a, " $= 13.9^k \times 4.0' = 55.6$

$\overline{117.8^{k'}}$

INTER. COLUMN (MOM. AT ℄ GIRD)

ABOVE b, $M = 22.7 \times 5.50 = 124.9$

BELOW b, " $= 27.9 \times 4.0 = 111.6$

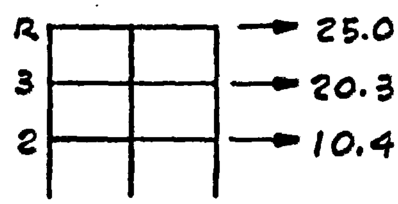
$\overline{236.5^{k}}$

GIRDER (MOM. AT ℄ COL.)

$M_a = 117.8 \quad M_b = \frac{236.5}{2} = 118.3$

$V = \frac{117.8 + 118.3}{24'} = 9.84^k$

* ESTIMATED LOCATION OF INFLECTION CONSIDERING FIXITY OF BASE.



FRAME FORCES

AT UPPER POINT OF INFLECTION,

$M = (25.0^k \times 18.6') + (20.3^k \times 6.7') = 601^{k'}$

$\text{AXIAL} = \frac{601^{k'}}{48} = 12.5^k$

AT LOWER p. i.

$M = 601 + (45.3 \times 9.5) + (10.4 \times 4.0) = 1073^{k'}$

$\text{AXIAL} = \frac{1073}{48} = 22.4^k$

PRELIMINARY DESIGN - CONT. FRAME 4

INTERIOR COLUMN

a. VERTICAL LOAD ON CENTER FRAME

ROOF DL: $0.0257^{KSF} \times 24' \times 32' = 19.8$

3RD FLR. $DL+LL = (0.0745 + 0.021) \times 24 \times 32' = 73.3$

2ND FLR. RED. LL ✓ $= 73.3$

$W = 166.4^{K}$

BY SYMMETRY, $M = 0$

b. SEISMIC LOAD, FROM P. 12

$P = 0$

$M = 27.9^{K} \times (4.00 - 0.75) = 90.7^{K}$ AT FACE OF GIRDER

c. VERTICAL + SEISMIC

$P = 166 + 0 = 166^{K}$

$M = 0 + 90.7 = 90.7^{K}$

USE AISC 7TH EDITION, P. 3-8

TRY W14 × 68, P. 3-16 $B_x = 0.195$

$$P_{EQUIV.} = \frac{166^{K} + 0.195(90.7 \times 12''/')}{1.33} = 284^{K}$$

FOR $h' = 9.5'$ $\frac{W14 \times 61}{I = 641}$ ALLOWS 334^{K}

EXTERIOR COLUMN

VERTICAL + SEISMIC

$P \simeq 166/2 + 22.4 = 105^{K}$

$M \simeq 50^{K \cdot 1}(est) + 90.7/2 = 95^{K \cdot 1}$

$\} P_{EQUIV.} = 246$

$\frac{W14 \times 48}{I = 485}$ ALLOWS 246^{K}

USE 14" COLUMNS FOR CONTROL OF DEFLECTIONS AND USE THE SAME SECTION FULL HEIGHT.

PRELIM. DESIGN - CONT. FRAME 4

GIRDER- 2ND FLOOR

VERTICAL LOAD AT CENTER COLUMN

$R = 0.08 \times 32 \times 24 = 61.4\%$ OR $23.1(1+74.6/50) = 57.5\%$

$RED.LL = 0.425 \times 50 = 21 PSF^{*}$ $(1 - 0.575 = 0.425)$

$w_{D+L} = (0.0746 + 0.021) \times 32' = 2.38 + 0.67 = 3.05^{K}/'$

$W_{D+L} = 3.05 \times 24' = 73.2^{K}$

$M \approx \frac{WL}{12} = \frac{73.2 \times 24}{12} = 146.4$

SEISMIC $M = \underline{118.3}$

VERT.+SEISMIC $M = 265$

USE AISC BEAM CHART, p. 2-92, 7TH ED, WITH $M = \frac{265}{1.33} = 199^{K'}$, AND UNBRACED LENGTH OF 6' FOR NEG. BENDING

$\frac{W18 \times 60}{I = 986}$ ALLOW $216^{K'}$

* ANSI A58.1

(R = LIVE LOAD REDUCTION)

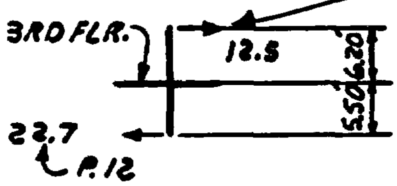
PRELIM. DESIGN - CONT. FRAME 4

GIRDER - 3RD FLOOR

VERTICAL LOAD - SAME AS 2ND, $M = 146^{K'}$

SEISMIC

$V_R = 25.0$ — 6.25^K TO EXT. COL. / 12.5^K TO INT. COL.



Σ COL. $M = (12.5^K \times 6.2') + (22.7 \times 5.5)$
$= 202^{K'}$

GIRDER $M = \dfrac{202}{2} = 101^{K'}$

VERT. + SEISMIC = $M = 146 + 101 = 247 \div 1.33 = 186^K$

W18x55 ALLOW 197

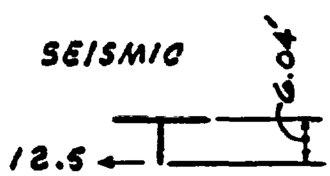
GIRDER - ROOF

VERTICAL LOAD

ROOF DL+LL = $32'(0.0257 + 0.020) = 0.82 + 0.64 = 1.46^{K/'}$

$W_D = 0.82 \times 24' = 19.7^K$ $M = \dfrac{19.7 \times 24'}{12} = 39.4^{K'}$

SEISMIC



GIRDER $M = \dfrac{12.5 \times 6.04}{2} = 37.8$

VERT. + SEISMIC = $M = 39.4 + 37.8 = 77.2 \div 1.33 = 58.0^{K'}$

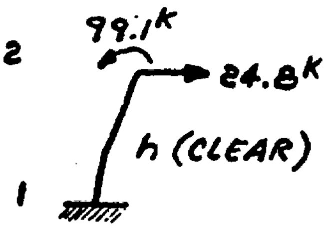
W14x22 ALLOW 58 OK FOR STRESS

USE W14x30 ALLOW 83 USE WIDER FLANGE FOR BETTER DETAILS

CHECK DRIFT - PRELIMINARY SIZES - FRAME 4

FOR DRIFT, USE FORCES ASSOCIATED WITH BARE-FRAME PERIOD, T = 0.88 SEC., AND MULTIPLY DISPLACEMENTS BY 1/K CHECK FIRST STORY

INTERSTORY DEFLECTION



P.9 P.12

$$M = \frac{127^K}{148} \times 111.6 = 99.1^{K'}$$

$$V = \frac{127}{148} \times 27.9 = 24.8^K$$

$$h = 10.67 - 1.50' = 9.17'$$

$$I = 641 \text{ IN}^4$$

INTERIOR COLUMN

$$EI\Delta = \frac{Vh^3}{3} - \frac{Mh^2}{2} = \frac{24.8 \times \overline{9.17}^3}{3} - \frac{99.1 \times \overline{9.17}^2}{2}$$

$$= 6374 - 4167 = 2207 \text{ K FT}^3$$

$$\Delta = \frac{2207 \times 144}{29{,}000 \times 614} = 0.0171 \text{ FT.}$$

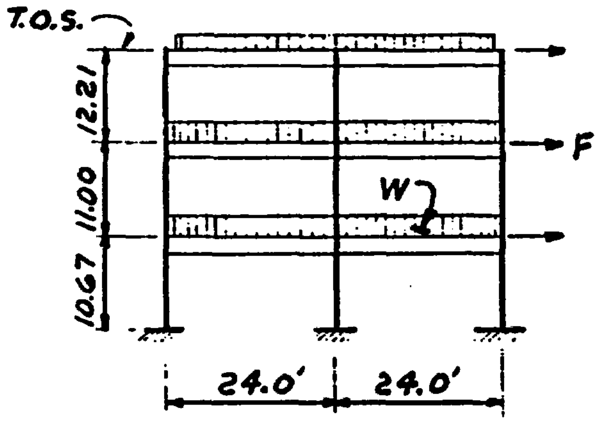
$$\frac{1}{K} \times \Delta = \frac{0.0171}{0.67} = 0.0255 \text{ FT.}$$

ALLOWED DRIFT = 0.005 × 11' = 0.055' > 0.0255

EVIDENTLY MEMBER SIZES COULD BE REDUCED BUT THIS WILL NOT BE ATTEMPTED UNTIL FINAL DESIGN. THE PRELIMINARY SIZES WILL BE USED FOR THE FRAME ANALYSIS.

FRAME ANALYSIS — FRAME 4

COMPUTER INPUT



KIPS, FEET, SECONDS

RIGID FRAME.
STEEL: E = 4,176,000 KSF
COLUMN BASES FIXED.

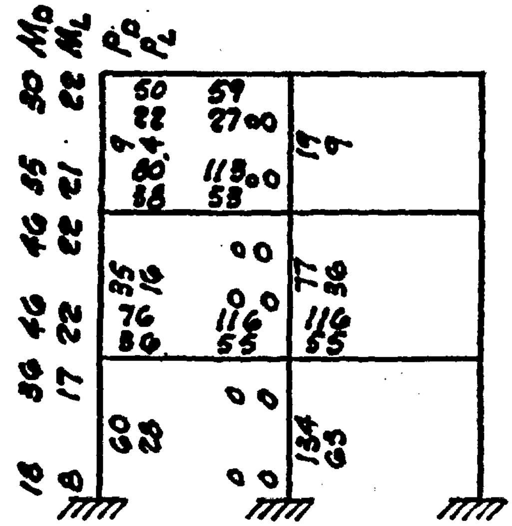
	EXT. COL.	INT. COL.
SIZE	W14x48	W14x61
I	0.0234	0.0309
A	0.0979	0.1243
Aw	0.0325	0.0365

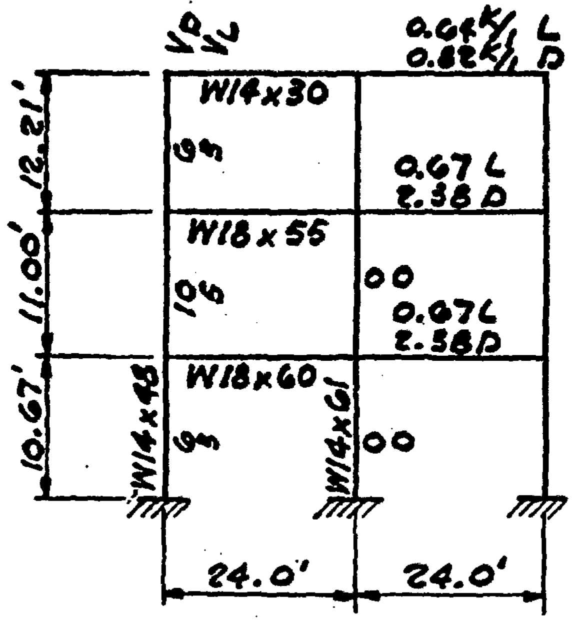
LEVEL	GIRDER						TRIB W	MASS = W/g	LATERAL FORCE
	SIZE	I	A	Aw	W_{DL}	W_{LL}			
R	W14x30	0.0140	0.0613	0.0260	0.82	0.64	187[k]	5.81	25.0
3	W18x55	0.0930	0.1125	0.0491	2.38	0.67	233	7.24	20.3
2	W18x60	0.0476	0.1229	0.0527	2.38	0.67	233	7.24	10.4
DATA FROM PAGE					14 & 15		5 & 6		11

TRIB. ROOF WT. 1/2 OF TOTAL SINCE DIAPH. IS FLEXIBLE
TRIB. FLOOR WT. IS 0.33 x TOTAL ACCORDING TO REL. RIG. OF FRAMES. $g = 32.2$ FT./SEC.^{2}

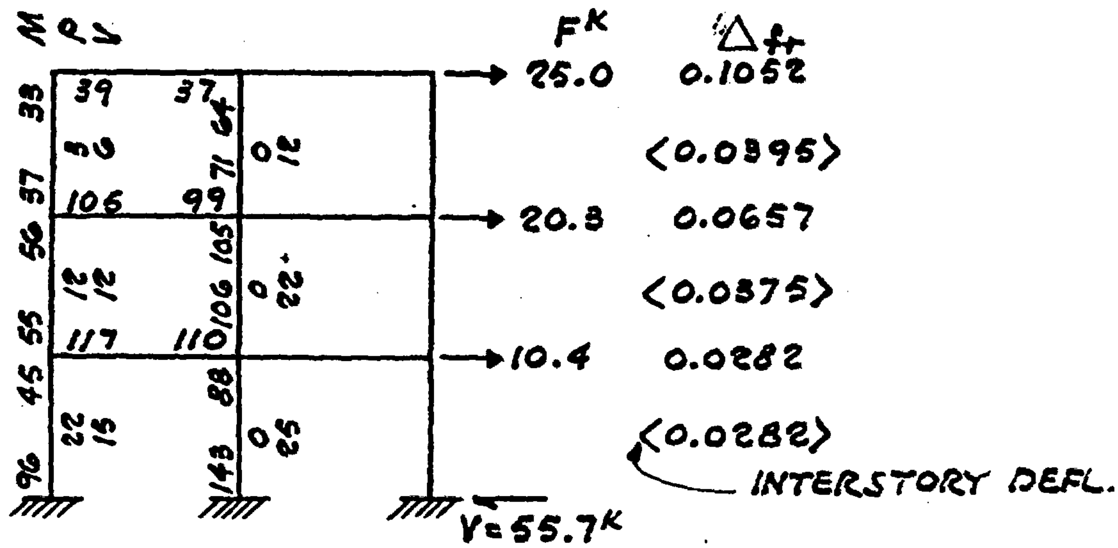
FRAME ANALYSIS - CONT. FRAME 4

COMPUTER OUTPUT





LIVE LOAD
DEAD LOAD



SEISMIC

RIGIDITY: $K = \dfrac{V}{\Delta_R} = \dfrac{55.7}{0.1052} = 529\ ^k/_{ft}$

FINAL DESIGN CRITERIA - FRAME 4

1. BUILDING PERIOD

   a. For drift use bare frame period of 0.86 sec obtained in a computer analysis assuming rigid diaphragms. See p. 34 for approximate calculation.

   b. For building design forces use whole building period of 0.69 sec (p.8)

2. DESIGN FORCES AND DEFLECTIONS

   a. Use forces obtained from computer analysis (p. 17 and 18) based on T = 0.69 sec base shear of 143^k (p. 9) and frame shear of 55.7^k (p. 11).

   b. Use deflections from this analysis in the drift calculations.

3. DRIFT (Refer to paragraphs 4-5c(1) and (2)

   a. Use bare-frame T = 0.86 sec (see 1a above).

   b. $C = 1/15\sqrt{0.86} = 0.0719$

   $CS = 0.0719 \times 1.5 = 0.108$

   $ZIKCS = 1 \times 1.0 \times 0.67 \times 0.103 = 0.0724$

   Base Shear $= 0.0724 \times 1789 = 130^k$

   Multiply deflections from frame analysis by the ratio $130/143 = 0.909$

   see page 18

   c. Maximum interstory defl. $= 0.909 \times 0.0395\ ft$
   $= 0.0359\ ft$

   Drift $= \frac{1}{K} \times D = \frac{0.0359}{0.67} = 0.054\ ft$

   Allowed drift $= 0.005 \times 11\ ft = 0.055\ ft$

FINAL DESIGN · CONT. FRAME 4
MEMBER STRESSES

(1) SAMPLE CALCULATION FOR 2ND FLR. GIRDER

	M_D	M_L	M_E	M_{D+L}	$\frac{M_{D+L+E}}{1.33}$
AT EXT. COL.	76	36	117	112	172
AT INT. COL.	116	55	110	171	211

DES. M = 211 K'　　UNBRACED LENGTH = 6'

W18x60 ALLOW M = 216 K'　AISC 7TH ED., P. 2-92

(2) SAMPLE CALCULATION FOR COLUMN,
SEE NEXT PAGE.

FINAL DESIGN - CONT. - FRAME 4
MEMBER STRESSES - CONT.

2.) SAMPLE CALCULATION FOR COLUMNS FIRST STORY AT BASE

$K_Y = 1.0$ (COLUMNS ARE BRACED BY BEAMS)

$K_X > 1.0$ USE NOMOGRAPH IN AISC, 7TH ED., P. 5-139

$G_B = 1.0$ FOR COLUMN RIGIDLY ATTACHED TO FTG.

$$G_A = \frac{\Sigma I_c/L_c}{\Sigma I_g/L_g}$$

$$G_A\Big|_{EXT.} = \frac{\frac{485}{11.0} + \frac{485}{9.92}}{0 + \frac{984}{24}} = 2.27$$

$$\underset{W18\times60}{}$$

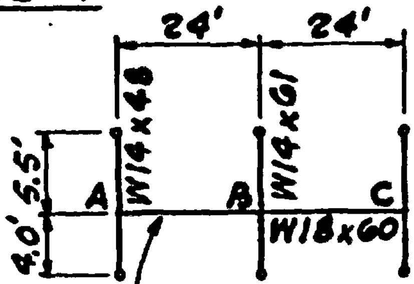
$$G_A\Big|_{INT.} = \frac{\frac{640}{11.00} + \frac{640}{9.92}}{\frac{984}{24} + \frac{984}{24}} = 1.50$$

$C_{MX} = 0.85$

	EXT.		INT.	
	W14x48		W14x61	
	A = 14.1, S = 70.2		A = 17.9, S = 92.2	
	P	M	P	M
D	60	18	134	0
L	28	8	63	0
D + L	88	26	197	0
E	22	96	0	148
$\frac{D+L+E}{1.33}$	83	92	148	108
f_a	5.89 KSI		8.27 KSI	
f_b	15.7 KSI		14.1 KSI	
K_x	1.47		1.39	
l	119"		119"	
r_x	5.85"		5.98"	
Kl/r_x	29.9		27.7	
F_a	20.0		20.1	
K_y	1.0		1.0	
l	119		119	
r_y	1.91		2.45	
Kl/r_y	62.3		48.6	
F_a	17.2		18.4	
F'_{ex}	167		194	
F_{bx}	24.0		24.0	
f_a/F_a	.342		.449	
f_b/F_b	.655		.586	
$\left\{C_{MX}/\left(1 - \frac{f_a}{F'_{ax}}\right)\right\} \times (f_b/F_b)$	.577		.520	
Σ	.919		.969	
$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} =$	.93		.97	

ALL SUMMATIONS < 1.0 OK

FINAL DESIGN · CONT. - FRAME 4
CONNECTIONS



SAMPLE CALC. FOR JOINT A

1. PLASTIC MOM. CAPACITY OF GIRDER.*

$$\frac{b}{2t_f} = 5.44 < 8.5 \text{ OK}$$

IGNORE SMALL HORIZ. P IN GIRDER:

$$\frac{d}{t} \le \frac{412}{\sqrt{F_y}} = 68.67 > 43.9 \text{ OK}$$

$M_p = 369\,K'$ > 298 OK

VERT. M $= 1.3 \times (76+36) = 146$

SEISMIC M $= \pm 1.3 \times 117 = \pm 152$

M $= 298\,K$

2. GIRDER CONNECTION.

VERT. LOAD : $V = (2.38 + 0.67)\frac{24'}{2} - \frac{171-112}{24} = 34.1\,K$

DESIGN V :

VERT. LOAD $1.3 \times 34.1 = 44.3$

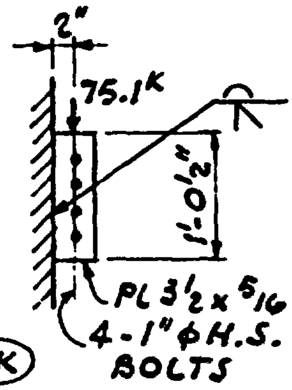
SEISMIC $\frac{2M_p}{L} = \frac{2 \times 369}{24} = 30.8$

DES. V $= 75.1^K$

BOLTS: USE 4 BOLTS; ASSUME 1"φ A490-F H.S. BOLTS ARE SELECTED, CONSIDERING FRAME AS A WHOLE.

ALLOW $V = 4 \times 1.7 \times 15.71$

$= 107^K > 75.1$ OK



SHEAR PL:

$Z = .3125 \times (12\tfrac{1}{2})^2/4 = 12.2 \text{ IN}^3$

$f = 75.1^K \times 2"/12.2 = 12.3 \text{ KSI} < 36$ OK

$v = \frac{75.1}{.3125 \times 12\tfrac{1}{2}} = 19.2 \text{ KSI} < 0.55 \times 36$ OK

*AISC SPEC. PART 2

FINAL DESIGN CONT. - FRAME 4

JOINT A - CONT.

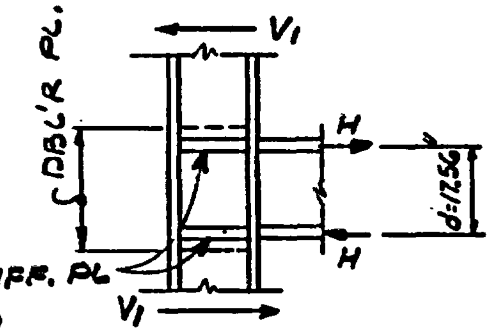
3. SHEAR IN COLUMN WEB.

ALLOW $V_u = 0.55 F_Y t_w d = 0.55 \times 36 \times 0.339 \times 13.81 = 92.7^k$

$V_1 = \frac{M_P}{h} = \frac{369^{k'}}{9.5'} = 38.8^k$

$H = \frac{M_P}{d} = \frac{369 \times 12}{17.56''} = 252$

$V_u = H - V_1 = 252 - 38.8$
$= 213 > 92.7$



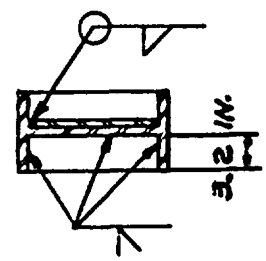
DOUBLER PL IS REQ'D
(OR A COLUMN WITH A THICKER WEB)

DOUBLER PL:

REQ'D $t = \frac{213}{92.7} \times 0.339 = 0.778''$ TOTAL

0.339 COL. WEB

USE 7/16" PL ⟶ 0.439 DOUBLER



SHEAR TRANSFER FORCE

$= \frac{.439}{.778}(213) = 120^k$

5/16" FILLET (E-70) $= \frac{120}{2 \times 1.7 \times 4.6} = 7.67''$

LENGTH OF PL = BM. DEPTH + 5K
$= 18 + (5 \times 1.25) = 24\frac{1}{4}''$

COLUMN STIFFENER PL =

COMPR. FLANGE $t = \frac{C_1 A_F}{t_b + 5K} = \frac{1 \times 7.558 \times .695}{.695 + (5 \times 1.25)} = .756 > .339$

REQ'D $A_{st} = [C_1 A_F - t(t_b + 5K)] C_2 =$
$= [1 \times 7.558 \times .695 - .339(.695 + 5 \times 1.25)] 1 = 2.89$

REQ'D $t = 2.89/2 \times 3.2 \text{IN} = 0.45 \text{IN}$ USE 1/2" PL

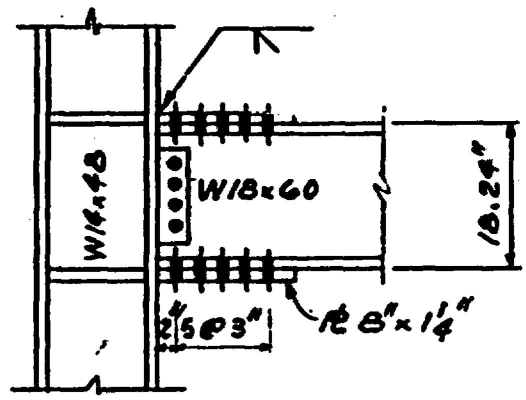
FINAL DESIGN-CONT. — FRAME 4

JOINT A - CONT.

ALTERNATE GIRDER FLANGE CONNECTION USING H.S. BOLTS

BM. $M_p = 369^{K'}$
ASSUME POINT OF INFLECTION AT MIDSPAN.

$L = 24'$ C. TO C.
$L_C = 22.85'$ CLEAR
$L_E = 20.02'$ TO LAST BOLT



DEVELOP PLASTIC MOMENT AT LAST BOLT: $V = \frac{2 \times 369}{20.02} = 36.9^K$

MOMENT AT FACE OF COL. $= \frac{36.9 \times 22.85}{2} = 422^{K'}$

SHEAR ON BOLTS $= \frac{422 \times 12}{18.24} = 278^K$

USE 12-1"φ A490-F H.S. BOLTS IN S.S.
ALLOW $12 \times 1.7 \times 15.71 = 320^K$ OK

FORCE IN PL $= \frac{422 \times 12}{18.24 + 1.25} = 260^K$

NET $A = [8 - (2 \times 1.063)] \times 1.25 = 7.35\ IN^2$

$F = \frac{260}{7.35} = 35.4$ KSI < 36 OK

CHECK COLUMN MOMENT: $M_p = 235^{K'}$

$M_{COL.} = \frac{369^K \times 24'}{2} = 443^K$, $V = \frac{443}{9.5} = 46.6^K$

COL. MOM. AT BEAM FLANGE
$= 46.6^K \times (5.5' - 0.75') = 221^{K'} < 235$ OK

LONGITUDINAL DIRECTION: STEEL BRACED FRAMES

LATERAL FORCES

BUILDING: A-3

DIRECTION: LONGITUDINAL

$Z = 1.0$, $I = 1.0$ $K = 1.00$

$C = 0.19$; $S = 1.5$ $CS = \cancel{0.89}$ → USE 0.14

$W = 1789$ KIPS

$$T = \frac{0.05h}{\sqrt{D}} = \frac{0.05 \times 34}{\sqrt{192}} = 0.123 \text{ SEC.}$$

$F_t = 0.07TV = 0\ V^*$

$F_x = (V - F_t)\frac{Wh}{\Sigma Wh} = 1.0\ V \frac{Wh}{\Sigma Wh}$

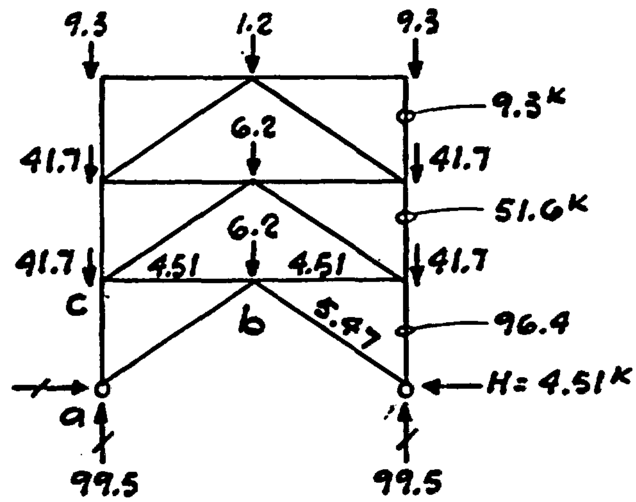
$V = ZIKCSW = 0.14\ W$
$= 250$ KIPS

* $F_t = 0$ WHEN $T \leq 0.7$ SEC.

LEVEL (1)	h FT. (2)	Δh FT. (3)	W KIPS (4)	ΣW (5)	(2)×(4) Wh (6)	Wh/ΣWh (7)	F KIPS (8)	Σ(7) V KIPS (9)	(3)×(9) ΔOTM K-FT. (10)	Σ(10) OTM K-FT. (11)	(8)÷(4) F/W (12)	(9)÷(5) V/ΣW (13)
						$F_t = 0$						
R	34		375	375	12,750	0.35	88				.235	.235
		12						88	1056			
3	22		707	1082	15,554	0.43	107			1056	.151	.180
		11						195	2145			
2	11		707	1789	7,777	0.22	55			3201	.078	.140
		11						250	2750			
										5951		
		Σ	1789		36,081	1.00	250					

BRACED FRAME - CONT.

VERTICAL FORCES ALL EXCEPT ROOF LL



TRIB. AREA AT COLUMN

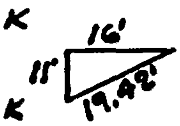
TRANSV. GIRDER : $9' \times 32' = 288$

EDGE BEAM $\left(\frac{32}{2} + \frac{16}{2}\right) \times 3' = 72$ } 360 SF

TRIB. AREA AT BRACE $16' \times 3' =$ 48 SF

LOADS AT COLUMNS			AT BRACES	
ROOF $(25.7\ PSF + 0) \times 360\ SF = 9.3^K$			$\times 48\ SF = 1.2^K$	
FLOOR $(74.5 + 36) \times 360\ SF = 39.8$	41.7^K		$\times 48\ SF = 5.3^K$	6.2^K
WALL $(5.3\ PSF \times 11') \times 32' = 1.9$			$\times 16' = 0.9$	

BRACE FORCE = $V = \frac{6.2}{2} = 3.1 \quad H = \frac{16'}{11'}(3.1) = 4.51^K$

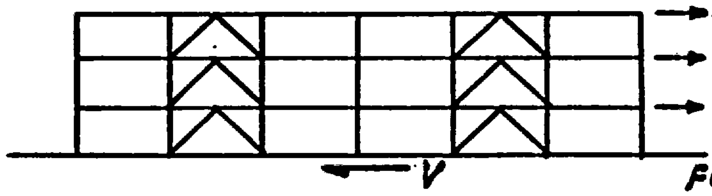


AXIAL FORCE = $\frac{19.4}{11}(3.1) = 5.47^K$

BRACED FRAME - CONT.

SEISMIC FORCES

FRAMES SHARE ROOF LOAD ACCORDING TO TRIBUTARY AREA. AT FLOORS THE FRAMES TAKE 0.59 x STORY FORCE (P. 11). MEMBERS OF BRACED FRAMES SHALL BE DESIGNED FOR 1.25 x THESE FORCES.



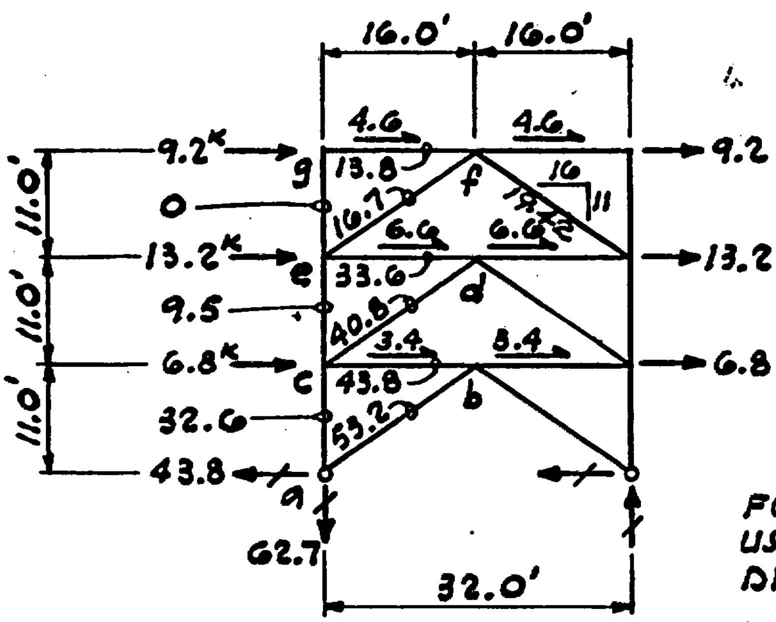
$\rightarrow 0.50 \times 88 \times 1.25 = 55^K$

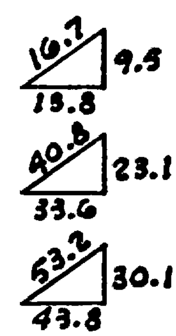
$\rightarrow 0.59 \times 107 \times 1.25 = 79^K$

$\rightarrow 0.59 \times 55 \times 1.25 = 41^K$

FOR MEMBERS, $V = 175^K$

APPLY 1/2 FORCE TO EACH BRACED BAY





FOR DEFLECTIONS USE THESE FORCES DIVIDED BY 1.25.

$$V = \frac{175}{1.25} = 140^K$$

TYPICAL BRACED BAY DESIGN FORCES

BRACED FRAME - CONT.

MEMBER DESIGN

COLUMN FLANGES IN PLANE OF FRAME

BENDING

WEAK DIRECTION = 0

STRONG DIRECTION:

GIRDER REACTION $= (.0745 + 0.036) \times 288 = 31.8^K$

e = HALF-COLUMN DEPTH = 7"

$$M = 31.8 \times \frac{7}{12} \times \underbrace{\frac{2}{3}}_{\text{CONTINUITY}} = 12.4^{K'}$$

AXIL

$$P = 99.5 \pm 32.6 = \begin{cases} 132^K \\ 67^K \text{ NO UPLIFT} \end{cases}$$

SIZE CHOSEN FOR ARCHITECTURAL COMPATABILITY WITH OTHER COLUMNS

TRY W14x43

$$\left.\begin{aligned} \frac{P}{A} &= \frac{132}{12.6} = 10.5 \text{ KSI} \\ \frac{M}{S} &= \frac{12.4 \times 12}{62.7} = 2.37 \text{ KSI} \end{aligned}\right\} \text{OK BY INSPECTION}$$

DIAGONAL BRACE

$$P = -5.47 \pm 53.2 = \begin{cases} -58.7 \\ +47.7 \end{cases}$$

STEEL TUBE $5 \times 5 \times \frac{1}{4}$ $P_a = 45$

$$\frac{P}{P_a} = \frac{58.7}{45} = 1.30 < 1.33$$

BRACED FRAME - CONT.

MEMBER DESIGN - CONT.

EDGE BEAMS - FULL LATERAL BRACING BY STEEL DECK. ASSUME WALL LATERAL LOADS TRANSMITTED DIRECTLY TO THE STEEL DECK: NO TORSION OR HORIZONTAL LOAD.

BEAM WITHOUT DIAGONAL BRACE $L = 32'$

BEAM MUST STILL CARRY VERTICAL LOAD EVEN IF THE BRACES SHOULD FAIL IN A LARGE EARTHQUAKE.

ROOF

$W = (0.0257 + 0.020) \times 3' \times 32' = 4.4^K$

W14x30

$$\Delta = \frac{5 \times 4.4 \times \overline{32}^3 \times 1728}{384 \times 29{,}000 \times 290} = 0.386'' = \frac{L}{995}$$

OK

FLOOR

$$\left.\begin{matrix} W = (0.0745 + 0.050) \times 3 \times 32 = 12.0 \\ \text{WINDOW } 0.0053 \times 11 \times 32 = 1.9 \end{matrix}\right\} 13.9^K$$

W18x40 AISC BEAM TABLE P. 2-38

$$\Delta = \frac{13.9}{36} \times 1.24 = 0.48''$$

BEAM WITH BRACE $L = 16'$

$$\left.\begin{matrix} \text{ROOF} & \text{USE W14x30} \\ \text{FLOOR} & \text{USE W18x40} \end{matrix}\right\} \text{FOR CONSISTENT DETAILS WITH UNBRACED BAYS}$$

VERT. $W = 0.183\text{KSF} \times 3' \times 16' = 8.78K$

$$M = \frac{8.78 \times 16'}{8} = 17.6^{K'}$$

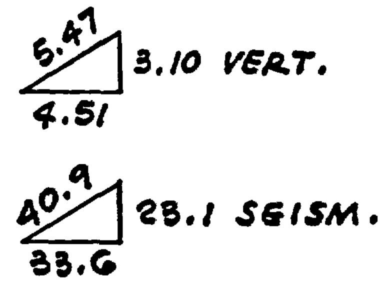
$$f = \frac{17.6 \times 12}{68.4} = 3.09 \text{ KSI} \quad \text{LOW}$$

$$\text{AXIAL } P = +4.51 \pm 43.8 = \begin{cases} +48.3 \\ -39.3 \end{cases}$$

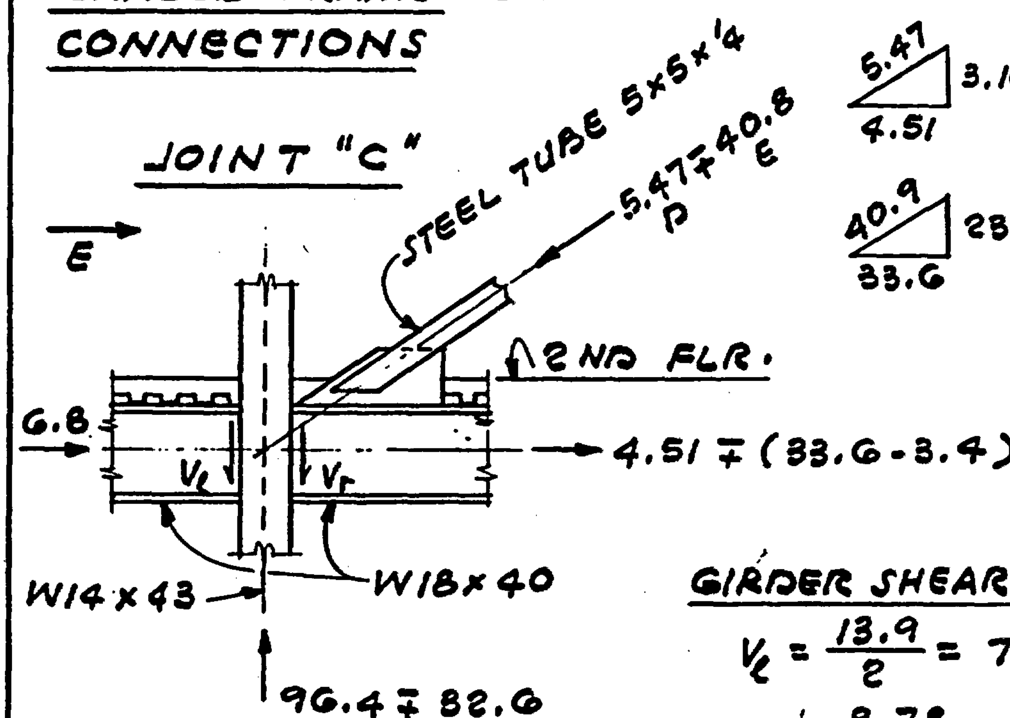
$$f_a = \frac{48.3}{11.8} = 4.09 \text{ KSI} \quad \text{LOW}$$

BRACED FRAME - CONT.

CONNECTIONS



JOINT "C"

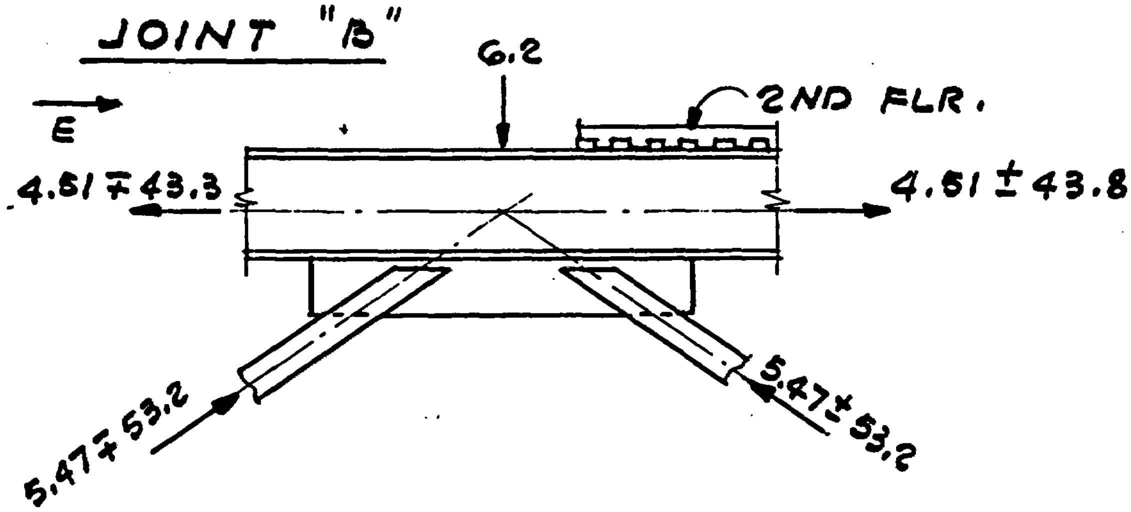


GIRDER SHEARS (VERT. LOAD)

$$V_l = \frac{13.9}{2} = 7.0^k$$

$$V_r = \frac{8.78}{2} = 4.4^k$$

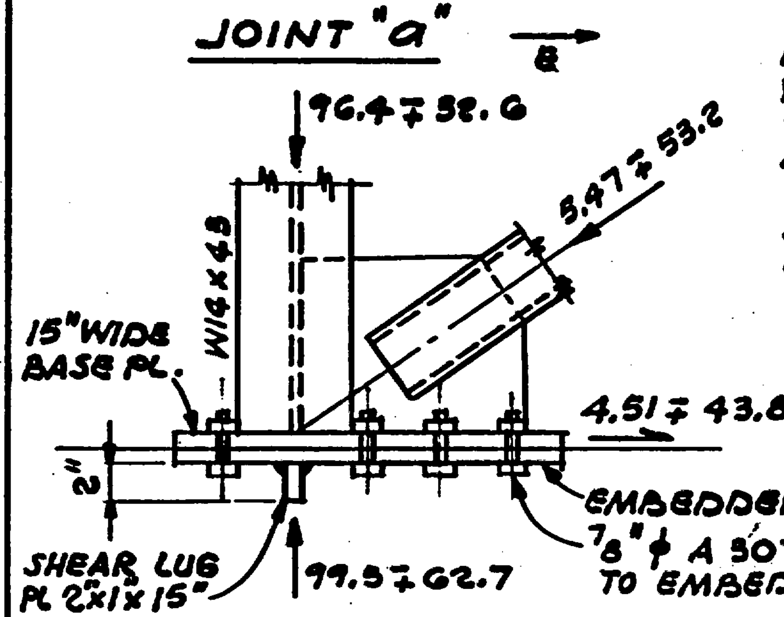
JOINT "B"



NOTE: CONNECTIONS SHALL BE DESIGNED FOR THE ABOVE FORCES WITHOUT THE USUAL ONE-THIRD INCREASE, OR SHALL BE DESIGNED TO DEVELOP THE FULL CAPACITY OF THE MEMBERS.

BRACED FRAME - CONT.

JOINT "a"



DESIGN FOUNDATION WITHOUT THE USUAL 1/3 INCREASE IN ALLOWABLE STRESS. USE ACI 318.77 APPENDIX B WITH FOUNDATION $f'_c = 3000$

VERT. BEARING ON CONC. $P = 99.6 + 62.7 = 162^K$

FOR FOUNDATION MUCH WIDER THAN THE LOADED AREA, ALLOWED BEARING STRESS $= 2 \times 0.3 f'_c = 1800$ PSI

REQ'D $A = \dfrac{162^K}{1.8 \text{ KSI}} = 90.0 \text{ IN}^2$.

MIN. LENGTH $= \dfrac{90.0}{15} = 6"$

SHEAR STUDS $V = 4.61 + 43.8 = 48.3^K$

USE $8 - \frac{7}{8}"\phi$ A 307 STUDS IN S.S.

ALLOW $8 \times 6.01 = 48.8^K$ OK

SHEAR LUG BECAUSE OF CONFINEMENT OF CONC. BY COL. ABOVE, USE ALLOW. BEARING = 1800 PSI

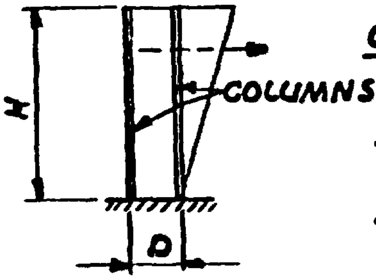
BEARING STRESS $= \dfrac{48,300}{15" \times 2"} = 1610$ PSI O.K.

BENDING : $M = 48.3^K \times 1" = 48.3$ K-IN $S = \dfrac{15" \times (1")^2}{6} = 2.5 \text{ IN}^3$

$f = \dfrac{48.3}{25} = 19.3 < 24$ O.K.

BRACED FRAME - CONT.

DEFLECTION

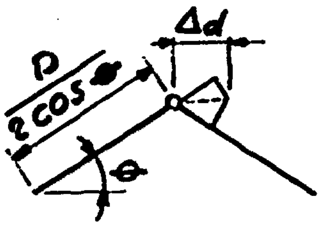


CHORDS : $\Delta_c = \dfrac{11\,VH^3}{60\,EI}$ *

$I = 2 \cdot A \cdot \left(\dfrac{D}{2}\right)^2 = \dfrac{AD^2}{2}$

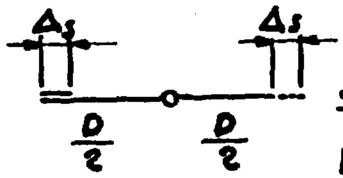
A = AREA OF ONE COLUMN

$\Delta_c = \dfrac{11\,VH^3}{60E\,\dfrac{AD^2}{2}} = \dfrac{11\,VH^3}{30\,EAD^2}$



DIAGONALS : $\Delta_d = \dfrac{P\,\dfrac{D}{2\cos\theta}}{EA\cos\theta} = \dfrac{f_d\,D}{2E\cos^2\theta}$

WHERE f_d IS THE STRESS IN THE STRUT



STRUTS : $\Delta_s = \dfrac{P \times \dfrac{D}{2}}{EA} = \dfrac{f_s\,D}{2E}$

WHERE f_s IS THE STRESS IN THE STRUT

$$\Delta = \Delta_c + \Delta_d + \Delta_s$$

$$= \frac{11\,VH^3}{30\,EAD^2} + \sum \frac{f_d\,D}{2E\cos^2\theta} + \sum \frac{f_s\,D}{2E}$$

$$\frac{2EA}{D} = \frac{11}{15} \cdot \frac{V}{A} \cdot \left(\frac{H}{D}\right)^3 + \sum \frac{f_d}{\cos^2\theta} + \sum f_s$$

IN COMPUTATION OF DEFLECTIONS USE STRESSES BASED ON MEMBER FORCES ÷ 1.25

* FORMULA FROM ROARK, FORMULAS FOR STRESS AND STRAIN

BRACED FRAME - CONT.

DEFLECTION - CONT.

CHORDS W14 × 34 $A = 10.0\ IN^2$

AT ROOF, $\frac{2E\Delta}{D} = \frac{11}{15} \cdot \frac{V}{A} \cdot \left(\frac{H}{D}\right)^3 = \frac{11}{15} \cdot \frac{87.5}{10.0} \left(\frac{33'}{32'}\right)^3 \div 1.25 = 5.63\ KSI$

AT 3RD & 2ND, $\frac{2E\Delta}{D} = 5.63\,(0.55,\ 0.17) = 3.10,\ 0.96$

DIAGONALS $\cos^2\theta = \left(\frac{16}{19.42}\right)^2 = 0.679$ 5×5×¼ $A = 4.54\ IN^2$

STORY	MEMBER F	$\frac{F}{1.25}$	A	f_d	$\frac{2EA}{D} = \frac{f_d}{\cos^2\theta}$
3	16.7^K	13.4^K	$4.54\ IN^2$	2.95	4.3 KSI
2	40.8	32.6	4.54	7.18	10.6
1	53.2	42.6	4.54	9.38	13.8

STRUTS

STORY		MEMBER F	$\frac{F}{1.25}$	A	$\frac{2EA}{D} = f_s$
3	W14×30	13.8^K	11.0^K	$8.85\ IN^2$	1.2 KSI
2	W18×40	33.6	26.9	11.80	2.3
1	W18×40	43.8	35.0	11.80	3.0

TOTAL DEFLECTION $E = 29{,}000\ KSI$, $D = 32' \times 12 = 384\ IN$

FLOOR	STORY	DIAG.		STRUTS		CHORDS		$\frac{2E\Delta}{D}$ KSI	Δ IN
	1	13.8		3.0					
2ND		13.8	+	3.0	+	0.96	=	17.8	0.12
	2	10.6		2.3					
3RD		24.4	+	5.3	+	3.10	=	32.8	0.22
	3	4.3		1.2					
ROOF		28.7	+	6.5	+	5.63	=	40.8	0.27

STIFFNESS OF WALL (2 FRAMES) $k = 2 \times \frac{87.5^K / 1.25}{0.27'' / 12} = 6222\ K/''$

FINAL PROPERTIES

TRANSVERSE PERIOD - FORMULA 3-3, CH. 3
FOR STIFFNESS USE THE F, Δ VALUES FROM THE ANALYSIS FOR FRAME A (P. 18). FOR MASS USE THE STORY WEIGHTS (P. 9)

LEVEL	W	F	Δ	$W\Delta^2$	$F\Delta$
R	375^K	25.0^K	0.1052 FT.	4.15	2.63
3	707	20.3	0.0657	3.05	1.33
2	707	10.4	0.0282	0.56	0.29
				7.76	4.25

FOR THE WHOLE BUILDING (W'S ABOVE) THERE ARE THREE FRAMES; SO USE $3 \times F\Delta = 12.75$

$$T = 2\pi\sqrt{\frac{W\Delta^2}{gF\Delta}} = \sqrt{\frac{7.76 \text{ K-FT.}^2}{32.2 \text{ FT/SEC}^2 \times 12.75 \text{ K-FT}}}$$

$= 0.86$ SEC. (BARE FRAME)

STIFFNESS (SEE P. 10 & 11)

IN FINAL DESIGN, $K_1 = 529$ K/FT (P. 18), REL. $K_1 = 1$
$K_A = 6222$ K/FT (P. 33), REL. $K_A = \frac{6222}{529} = 11.8$, SAY 12

FOR FRAME A OR C,
$Kd = 12 \times 24 = 288$, $Kd^2 = 6912$, $\Sigma Kd^2 = 32,256$

TORSIONAL SHEAR $= \frac{288}{32,256} \times 9.6 F_L = 0.0857 F_L$

DESIGN BASED ON $0.09 F_L$ IS STILL O.K.

DESIGN EXAMPLE: A-4

BUILDING WITH A DUAL BRACING SYSTEM:

Description of Structure. A two-story Office Building in Zone 4 with a complete reinforced concrete vertical load-carrying space frame. The lateral forces are resisted by a dual bracing system consisting of concrete ductile moment resisting space frames and concrete shear walls. The structural concept is illustrated on Sheet 2. The East-West direction is considered.

Construction Outline.

Roof:
- Built-up, 5-ply.
- Concrete joists and girders.
- Suspended ceiling.

2nd & 3rd Floors:
- Concrete joists and girders.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Bearing walls in concrete and non-bearing, non-shear insulated metal panels.

Partitions:
- Non-structural removable drywall, except concrete as structurally required.

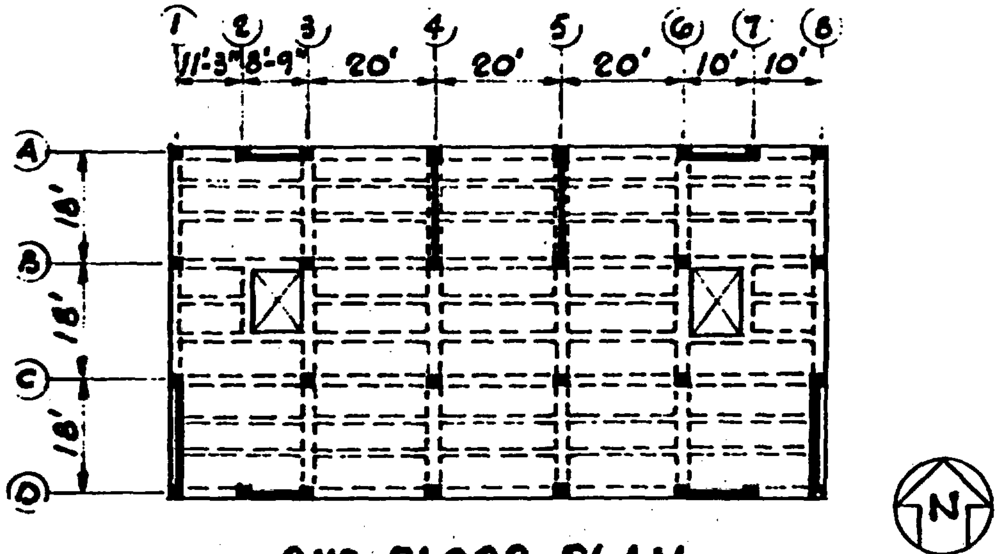
Design Concept. The structure is a dual bracing system meeting the requirements for a K-factor of 0.80 as follows: (1) the concrete shear walls are capable of resisting the total required lateral force; (2) the ductile moment resisting concrete space frame is capable of resisting not less than 25% of the total required lateral force; and (3) it is assumed, for purposes of the example, that a rigidity analysis of the walls and frames, considering their interaction, would show that they would not be called upon to resist forces higher than those obtained under (1) and (2). The roofs and floors form rigid diaphragms, and the seismic loads to the frames are proportioned according to their stiffnesses and the loads to the walls according to theirs. The building is assumed to be symmetrical about both axes so that only accidental torsion is involved. Special boundary conditions are required for the shear walls. See Chapter 6, paragraph 6-3a(1)(D).

Discussion. Vertical and lateral forces are pre-computed. (See Example A-5 for a typical computation.) The shear walls in the south wall (Line D) are designed for the given lateral forces. The seismic frames would be designed for 25% of these forces, using the methods of Example A-2. Deformation compatibility is investigated for the nonseismic frames.

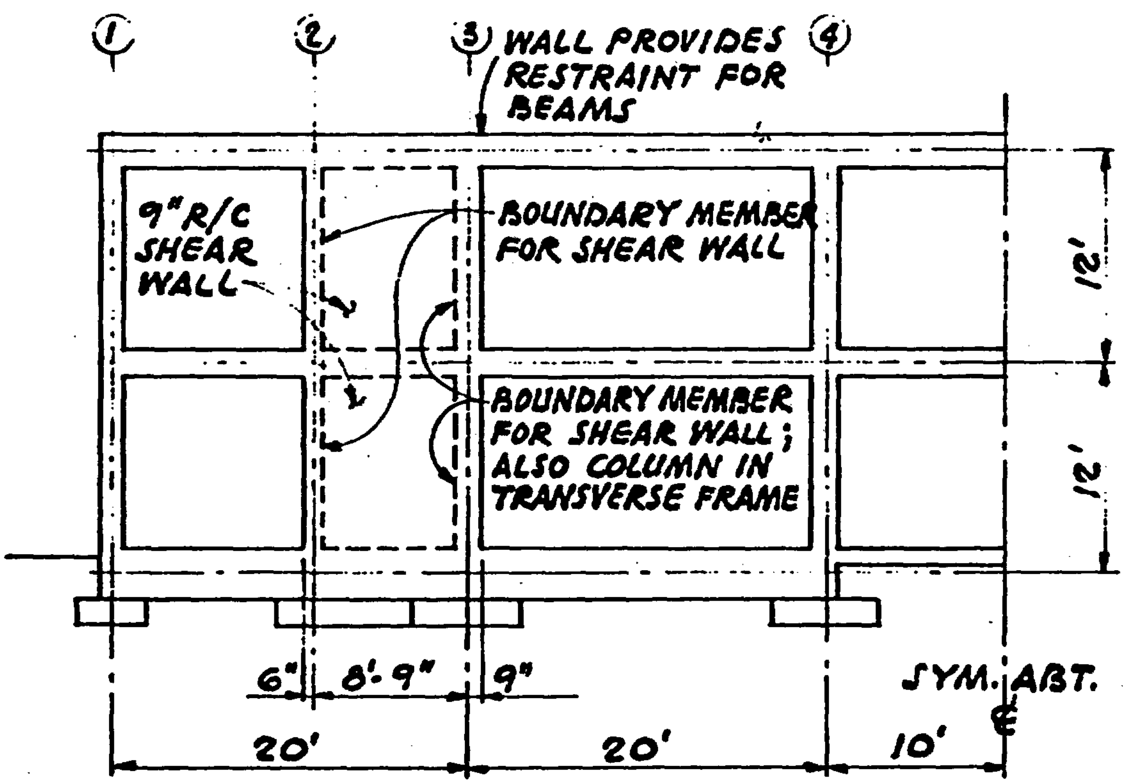
Materials.

Concrete f'_c = 3,000 psi.

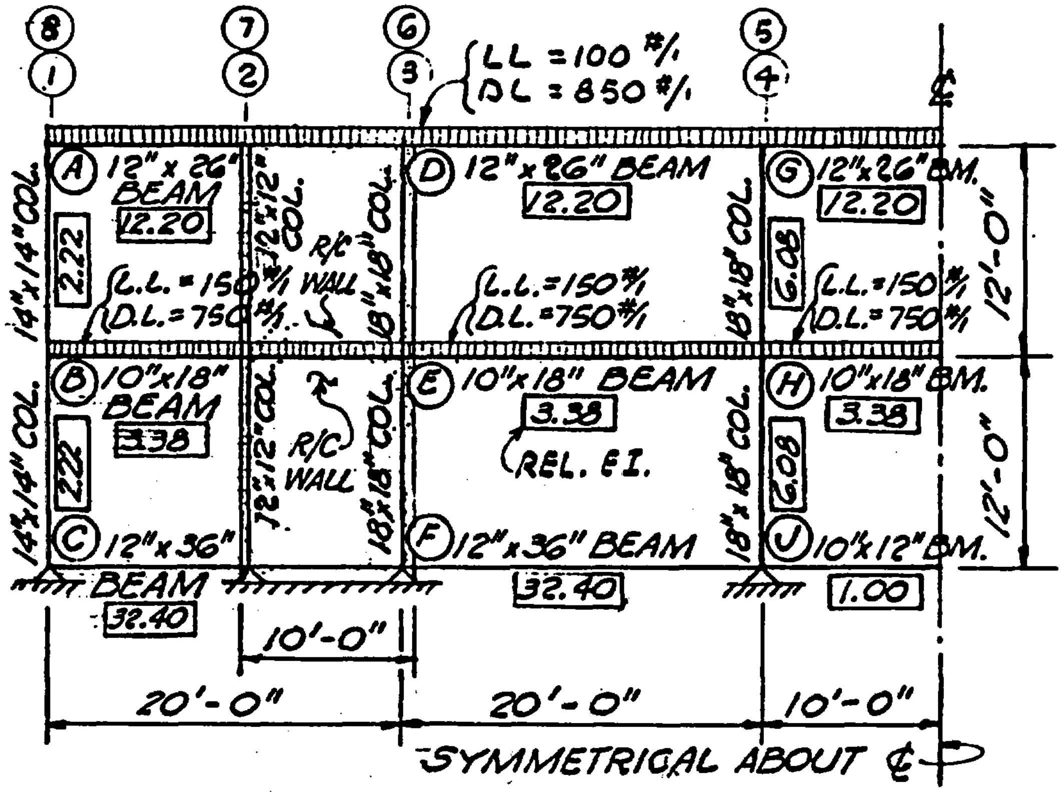
Reinf. Steel f_y = 60,000 psi.



2ND FLOOR PLAN



HALF ELEVATION OF SOUTH WALL-LINE D



MEMBERS AND LOADS - SOUTH WALL - LINE D

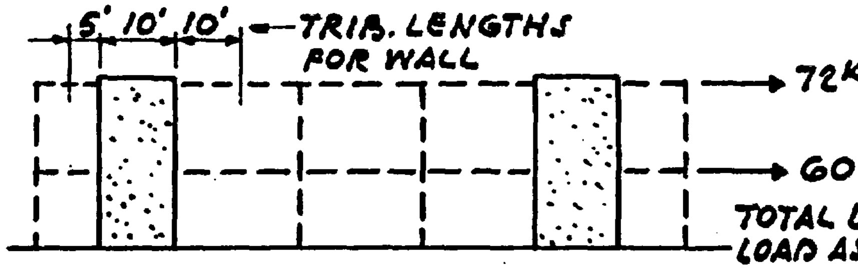
FRAME PROVISIONS

The two interior frames (Lines B and C) will be designed as ductile moment resisting space frames to carry 25% of the total required lateral force. See Example A-2. This example will deal only with Lines A and D which have shear walls that carry 100% of the lateral force. Deformation compatibility (para. 3-3(J)1d) must be investigated for the vertical load-carrying frames on Lines A and D (see p. 11).

As an alternate, the interior frames could be designed for vertical load only (with an investigation for deformation compatibility), and the lateral forces would be carried by ductile moment resisting space frames on Lines A and D. In these frames there is a choice concerning the columns on Lines 2 and 7: the columns may be treated as columns with adjacent girders of 10' span, or they may be treated as boundary members for the shear walls. In the latter case, the girders must still be designed, together with the columns, for the actual 10' spans, but they must also be designed to span 20' from 1 to 3 and from 6 to 8 in case the shear walls and boundary members fail.

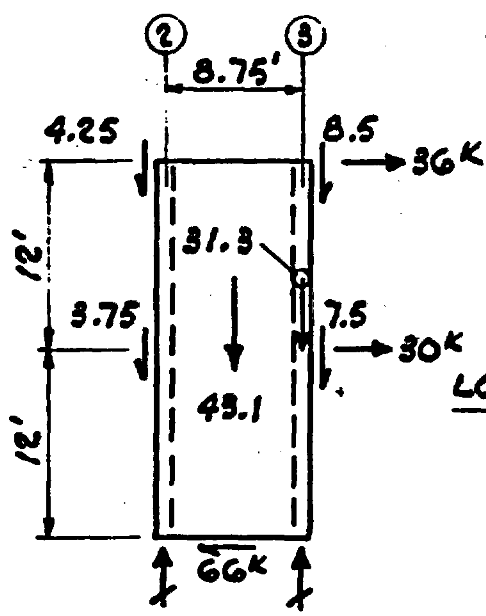
SHEAR WALL ANALYSIS

SHEAR WALLS ARE DESIGNED FOR 100% OF THE TOTAL REQUIRED LATERAL FORCE. DESIGN FORCES FOR THE SOUTH WALL ARE SHOWN.



TOTAL LATERAL LOAD ASSIGNED TO SOUTH WALL. EACH SHEAR WALL TAKES HALF.

SOUTH WALL ELEVATION



SOUTH WALL BEAM REACTIONS		
FLAT LOAD, EDGE BEAM & WALL	LEFT 5' TRIB.	RIGHT 10' TRIB.
ROOF:		
DL @ 0.85 K/'	4.25	8.5
LL @ 0.10	0.50	1.0
FLOOR:		
DL @ 0.75 K/'	3.75	7.5
LL @ 0.15	0.75	1.5

LOAD TRIBUTARY DIRECTLY TO WALL

ROOF	0.85 x 10 =	8.5
FLOOR	0.75 x 10 =	7.5
WALL	10 x 24 x 0.113 =	27.1
	TOTAL DL	43.1^K
FLOOR LL	0.15 x 10 =	1.5^K

8.0	16.0	
21.6	21.6	
	31.3	
29.6	68.9	VERT. DL
$\pm 140^K$	$\pm 140^K$	SEISMIC (P. 5)

LINE 3 GIRDER REACTION

ROOF + FLOOR DL = 31.8^K
FLOOR LL = 6.0^K
(FROM TRIB. AREA TO GIRDER C-3 - D-3 CALCULATION NOT SHOWN)

SHEAR WALL ANALYSIS - CONT'D

OVERTURNING

$$M_{OT} = (36^K \times 24') + (30 \times 12) = 1224^{K'}$$

$$F_{OT} = \pm \frac{1224}{8.75} = \pm 140^K$$

UPLIFT

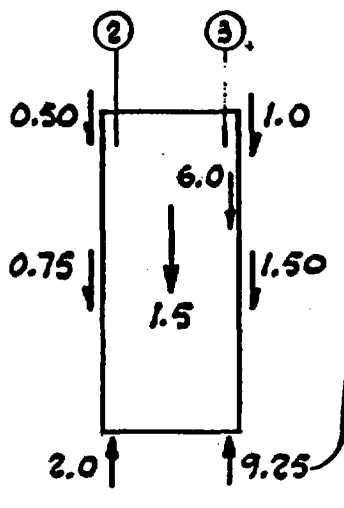
CALCULATE MAX. NET UPLIFT USING 0.9 x DEAD LOAD

$$P = 0.9(29.6^K) - 140^K = -113^K$$

THIS COULD BE REDUCED BY INCLUDING FOUNDATION DEAD LOAD, OR BY WIDENING THE WALL (IF ARCHITECTURALLY ACCEPTABLE).

THE NET UPLIFT FORCES COULD BE TAKEN BY DRILLED PIERS OR ROCK ANCHORS IF APPROPRIATE.

MAX. COMPRESSION



VERT. LL

VERT. DL	68.9	P. 4
SEISMIC	140	P. 4
VERT. LL	9.3	
	118.2^K	

APPLY TO FOOTING, LINE 3

SHEAR WALL DESIGN

WALL SHEAR $V_u = 2.0E$

$V_u = 2.0\left(\frac{72+60}{2}\right) = 132$ $\quad A_c = 9'' \times 10' \times 12''/_1 = 1080 \text{ IN}^2$

SHEAR CARRIED BY CONC.= $\quad v_u = \frac{132{,}000}{0.85 \times 1080} = 144 \text{ PSI}$

$v_c = 2\sqrt{f_c'} = 2\sqrt{3000} = 110 \text{ PSI}$ $\quad < 8\sqrt{f_c'}$ OK

$V_c = 0.110 \times 1080 = 119^K$

SHEAR CARRIED BY REINF. $V_u' = \frac{V_u}{\phi} - V_c = \frac{132}{0.85} - 119 = 36.3^K$

$A_v = \frac{V_u' s}{f_y d}$ $\quad s = \frac{A_v f_y d}{V_u'}$

Try #4 @ 18" CC EACH WAY EACH FACE

REQ'D $s = \frac{2(0.20)(60)(9.25 \times 12)}{36.3} = 73''$

$A_{s\,MIN.} = 0.0025bd = 0.0025 \times 9 \times 12 = 0.27 \text{ IN.}^2/\text{FT.}$

A_s PROVIDED $= \frac{12}{18} \times 2 \times 0.20 = 0.27$ OK

ALLOWABLE SHEARING STRESS $= 2\sqrt{f_c'} + \rho f_y$

$= 110 + (0.0025 \times 60{,}000) = 110 + 150 = 260 > 144$

REQ'D MIN. SPACING $= \frac{d}{3} = \frac{10'}{3}$ OR 18"

OR $3b = 27''$ $\quad$ ACI 11.10.9.5

SHEAR-FRICTION AT CONSTRUCTION JOINT AT BASE

$A_v = \frac{V_u}{\phi f_y \mu} = \frac{132}{0.85 \times 60 \times 0.60} = 4.30 \text{ IN.}^2$ ACI 11-7 {Para. 6-3c

$4.30 \div 10 = 0.43 \text{ IN}^2/\text{FT.} < 0.27 \text{ IN.}^2/\text{FT}$ N.G.

∴ PROVIDE INTERM.
#4 DOWELS @ 18" O.C.

SHEAR WALL DESIGN - CONT'D.

VERTICAL BOUNDARY MEMBER - LINE 2

VERTICAL LOADS: 12"x12" COL.

FOR MAX. TENSION,

$$REQ'D\ P_U = 0.9D - 1.40E$$
$$= 0.9(29.6) - 1.4(140) = -169^K$$

FOR MAX. COMPRESSION

$$REQ'D\ P_U = 1.4(D+L) + 1.4E$$
$$= 1.4(29.6+2.0) + 1.4(140) = +240^K$$

FOR TENSION ON COL. CORE:

$$\frac{P_U}{\phi} = \frac{169}{0.90} = 188^K$$

$$A_s = \frac{188}{60} = 3.13^{\square''}$$

FOR COMPRESSION:

$$P_U = 240^K \qquad \gamma = \frac{12-4.38}{12} = 0.635$$

USE $\gamma = 0.60$

$$\left.\begin{array}{l} \dfrac{P_U}{A_G} = \dfrac{240}{12 \times 12} = 1.67 \\ \dfrac{e}{h} = 0.10\ MIN. \end{array}\right\}$$ FROM SP-17A (78) CHART R3-60.60

$$REQ'D\ \rho_g = 0.010$$
$$A_s = 0.01(12 \times 12) = 1.44^{\square''} < 3.13$$

TENSION CONTROLS:

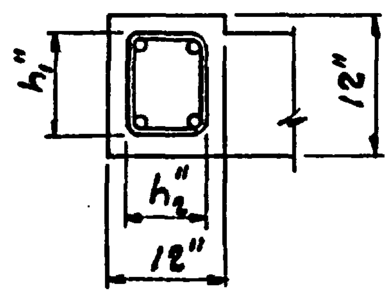
PROVIDE 4-#8 (3.16$^{\square''}$)
VERTICAL COLUMN CORE REINFORCEMENT

SHEAR WALL DESIGN - CONT'D
VERTICAL BOUNDARY MEMBERS - CONT'D

SPECIAL TRANSVERSE REINFORCEMENT:

FOR #4 HOOPS : $h_1'' = 12 - 3.0 = 9''$

$A''_{SH} = .40^{\square''}$ $\quad h_2'' = 12 - 3.0 = 9''$



FROM CHAPTER 7

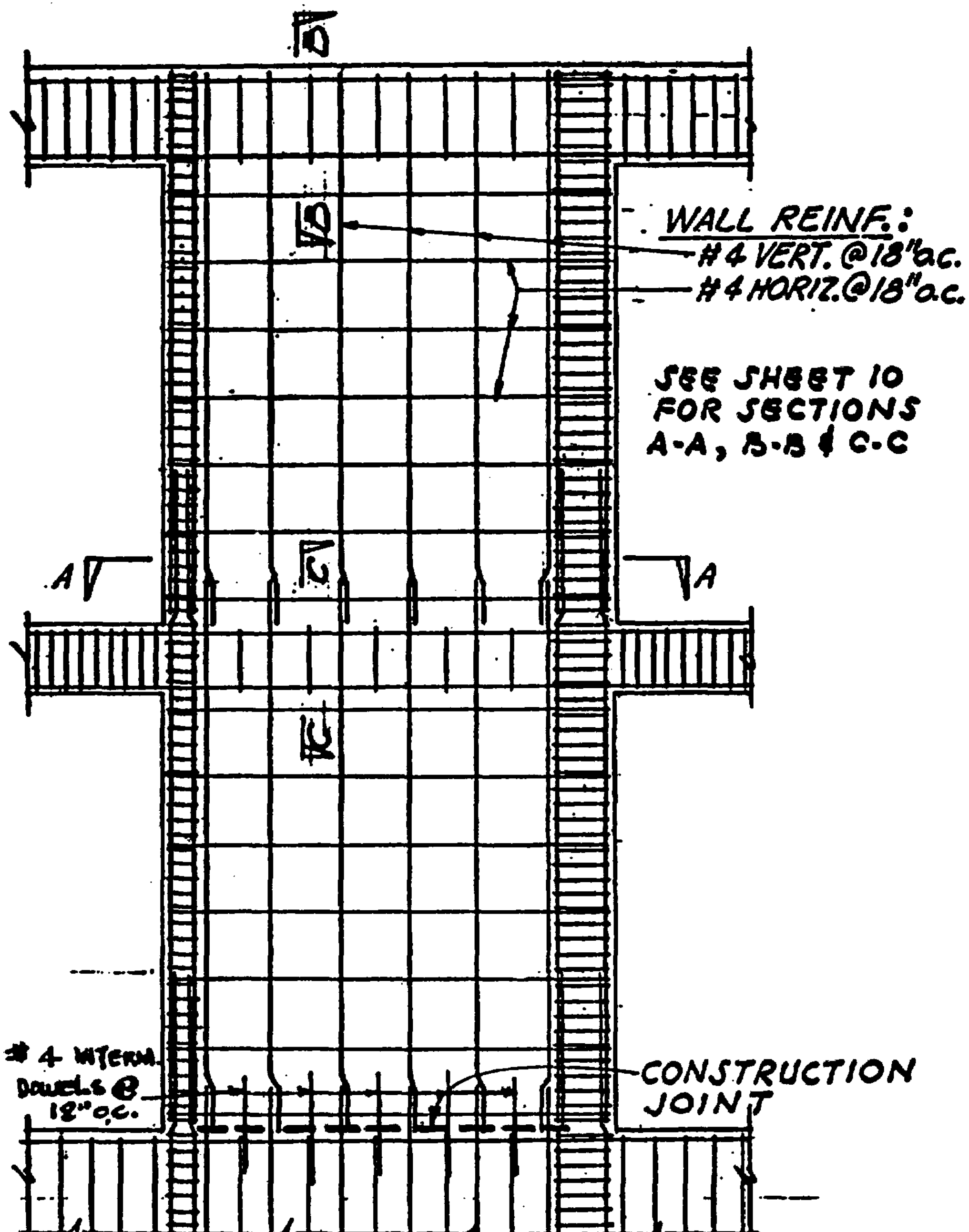
$$a = \frac{A''_{SH}}{0.30\ h'' \frac{f'_c}{f''_{yh}}\left[\frac{A_g}{A_c} - 1\right]}$$

$$= \frac{.40}{0.30(9.0)\frac{3}{60}\left[\frac{12 \times 12}{9 \times 9} - 1\right]} = 3.81''$$

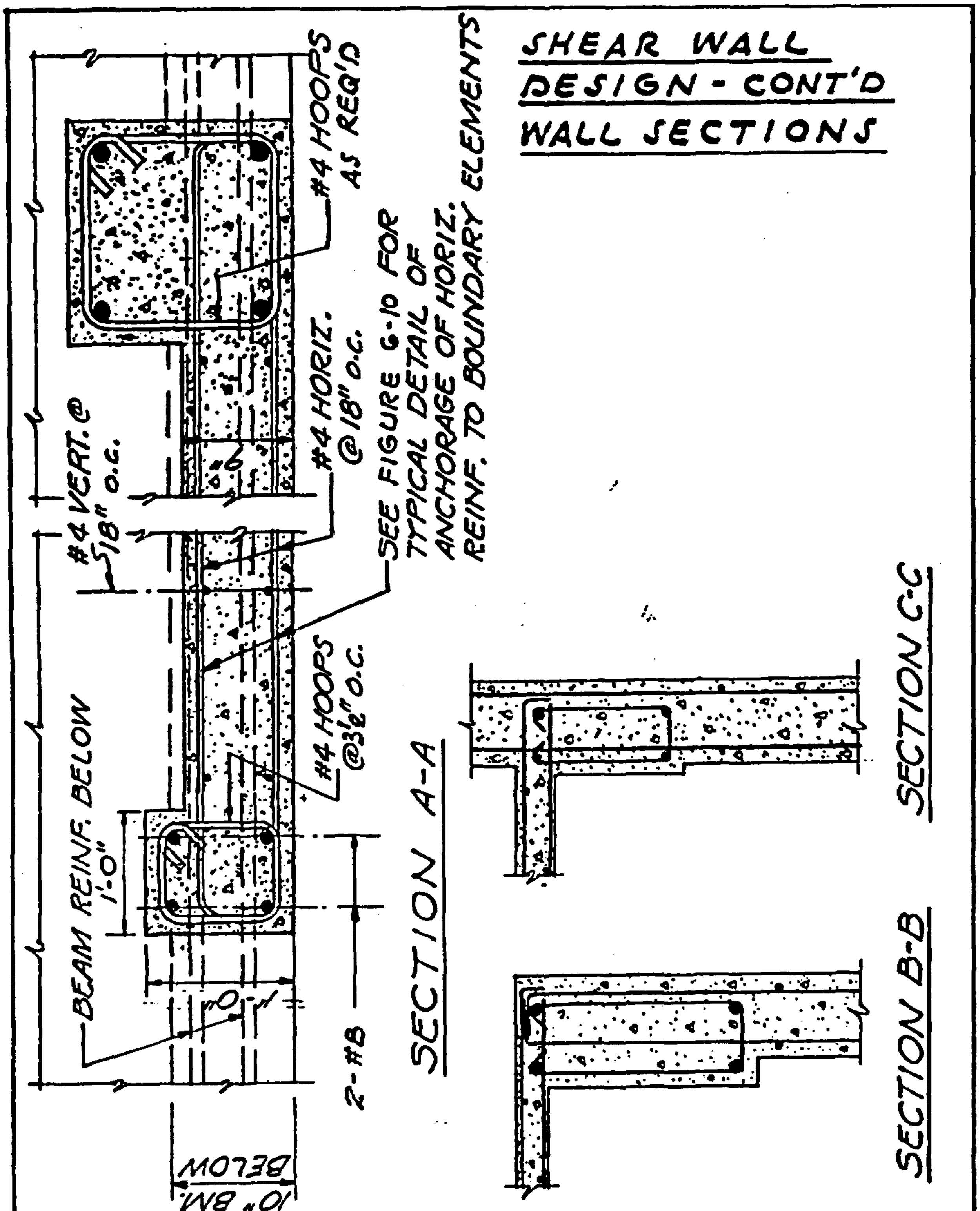
OR $$a = \frac{A''_{SH}}{0.12h''\ \frac{f'_c}{f''_{yh}}} = \frac{.40}{0.12(9)\frac{3}{60}} = 7.41''$$

$S_{MAX} = 3.81''$ $\quad$ ∴ USE #4 HOOPS @ $3\frac{1}{2}''$ O.C. THRU-OUT LENGTH OF COL. CORE

SHEAR WALL DESIGN - CONT'D

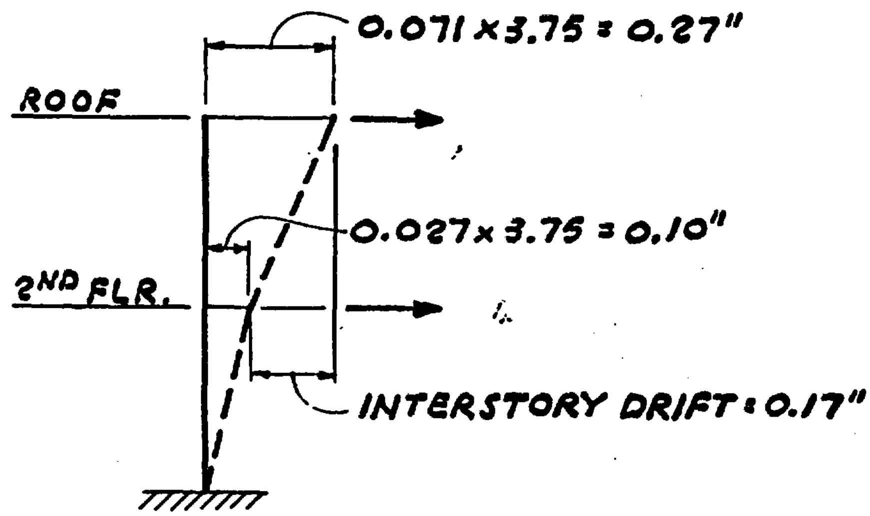


WALL ELEVATIONS



Example A-4 10 of [illegible] Concrete Frames, Concrete Walls

<u>Deformation Compatibility, (3/K) Times Deflection</u>. In this example, the shear walls (with vertical boundary members) on Lines A and D and the frames on Lines B and C are designed to resist the seismic forces. The framing members on Line A and D (other than the shear wall vertical boundary members) are not part of the lateral force resisting system; therefore, they will be investigated for deformation compatibility (para. 3-3(J)1d). When the lateral forces shown on page 4 are applied to the structure, the lateral displacement is 0.071 inch at the roof and 0.027 inch at the floor level. The framing members on Lines A and D must be investigated for 3/K (3/0.8 = 3.75) times these displacements. Refer to SEAOC Commentary, p. 45-C to p. 47-C. Also, see Design Example A-7, p. 8 and 9.



The resulting member forces are combined with the forces due to vertical gravity loads. In this example, the resulting stresses are within the elastic capacity of the members and the P-Δ effects are negligible. Therefore, the requirements for deformation compatibility are satisfied.

DESIGN EXAMPLE: A-5

BUILDING WITH A DUAL BRACING SYSTEM:

Description of Structure. A three-story Administration Building in Zone 4 with dual bracing system consisting of a ductile moment resisting space frame in structural steel and concrete shear walls. The structural concept is illustrated on Sheets 2, 3, and 4.

Construction Outline.

Roof:
- Built-up, 5-ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd & 3rd Floors:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

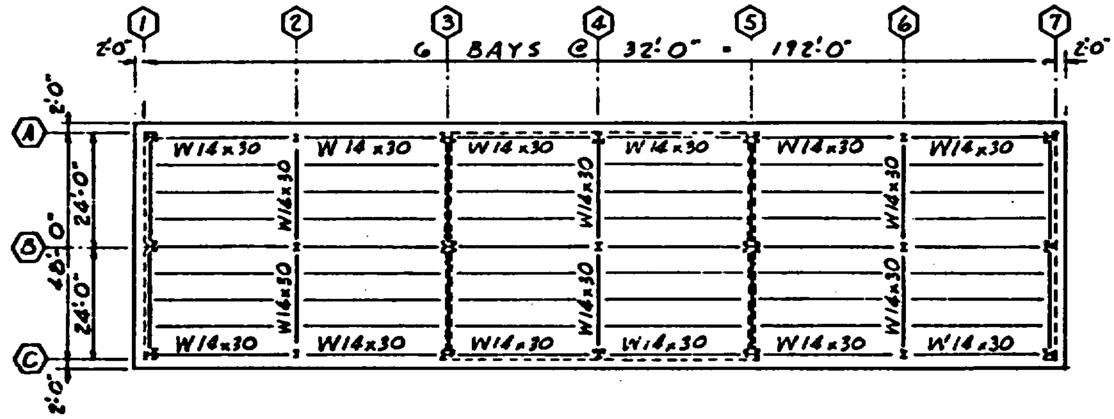
1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Bearing walls in concrete and non-bearing, non-shear insulated metal panels.

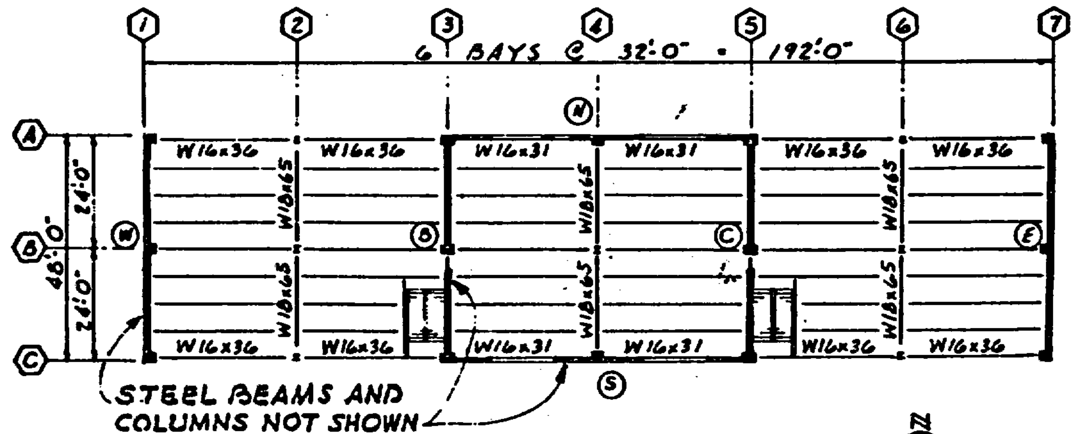
Partitions:
- Non-structural removable drywall, except concrete as structurally required.

Design Concept. Since the structure is a dual bracing system with a ductile moment-resisting space frame in structural steel capable of resisting not less than 25 percent of the required lateral force and concrete shear walls capable of resisting the total required lateral force, the K-factor is 0.80. The metal deck roof system forms a flexible diaphragm; therefore the roof loads are distributed to the frames and/or shear walls by tributary area rather than by stiffnesses. The metal deck with concrete fill systems for the floors form rigid diaphragms and the seismic loads are proportioned to the frames and/or shear walls by their stiffnesses.

Discussion. Portions of the exterior walls are insulated steel sandwich walls, not capable of acting as shear walls. Other portions of the exterior walls are of reinforced concrete. Two interior concrete shear walls are provided to the roof to support the flexible roof diaphragm and to reduce north and south wall deflections. The rigidity of the steel frame as compared to the shear walls is insignificant; therefore, the analysis of the total structure assumes that all lateral forces go to the shear walls using a K-factor of 0.80. The calculations for distribution of forces to the shear walls is not given here since these follow procedures given in Example A-1. Calculations are given for the amount of shear to each floor for 100% of the total base shear to the shear walls and the amount of shear to each floor due to the requirement of 25% of the total base shear to the frame alone.



ROOF PLAN



2ND & 3RD FLOOR PLAN

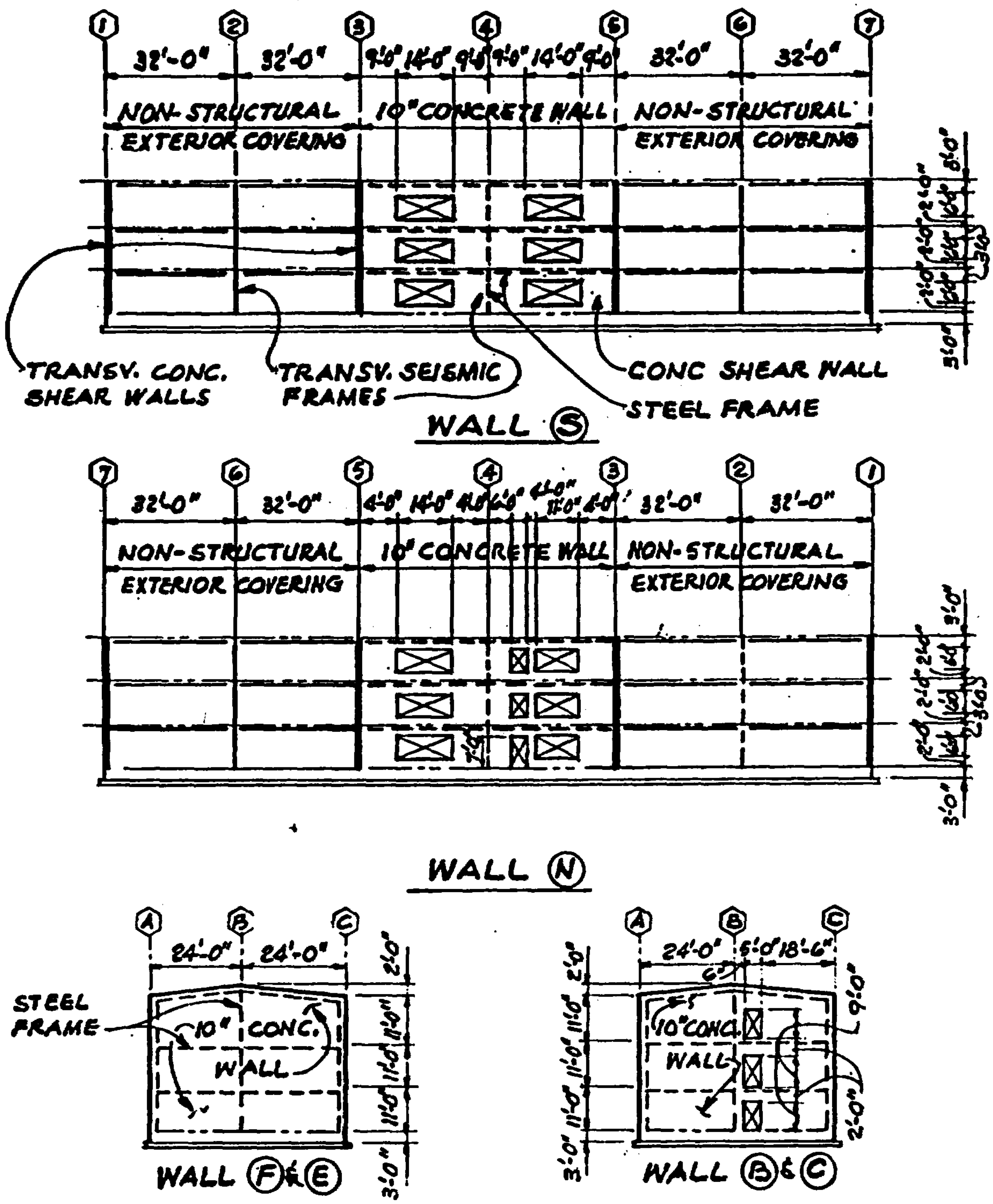
LOADS.

ROOF:

5 PLY ROOFING	6.0 #/□'
1" INSULATION	1.5
STEEL DECK	2.3
STEEL PURLINS	3.7
STEEL GIRDERS AND COLUMNS	1.2
CEILING	10.0
MISCELLANEOUS	1.0
DEAD LOAD	25.7
ADD FOR SEISMIC LOAD:	
PARTITIONS	10.0
TOTAL FOR SEISMIC =	35.7 #/□'

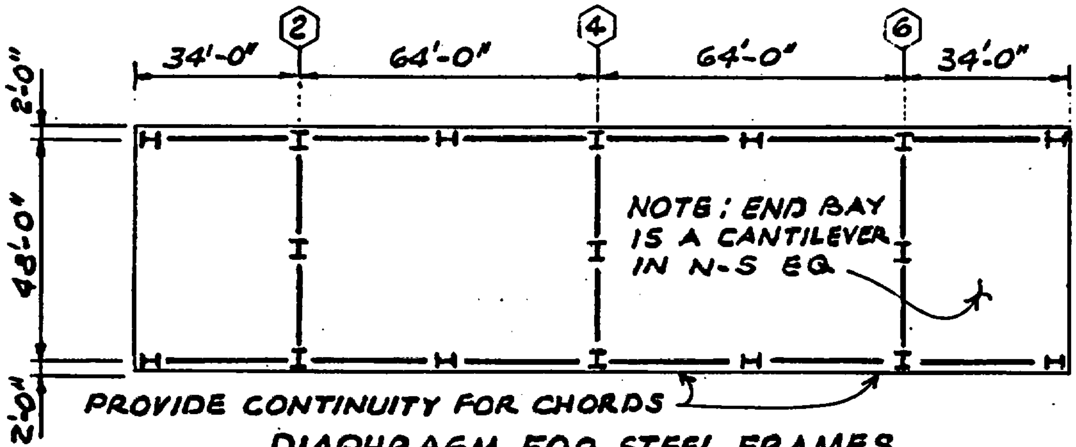
2ND & 3RD FLOORS:

FINISH	1.0 #/□'
STEEL DECK	3.1
CONCRETE FILL	32.0
STEEL BEAMS	5.9
STEEL GIRDERS AND COLUMNS	1.5
PARTITION	20.0
CEILING	10.0
MISCELLANEOUS	1.0
DEAD LOAD	74.5 #/□'
LIVE LOAD	50.0 #/□'

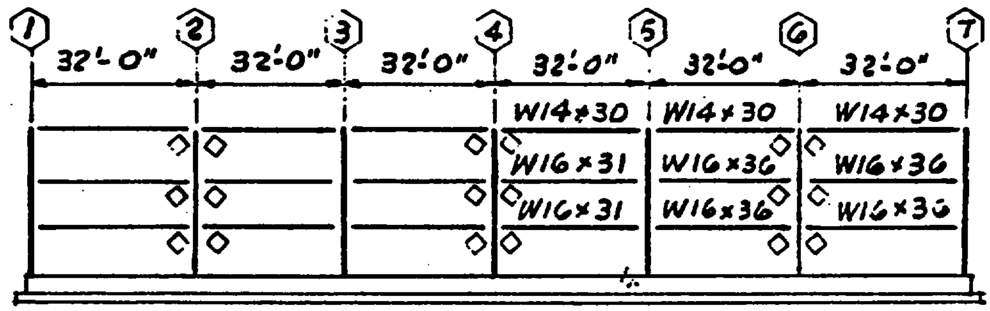


ELEVATIONS OF CONCRETE SHEAR WALLS

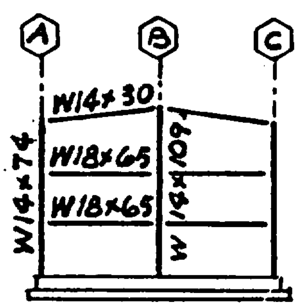
SEE SHT. 4 FOR STEEL FRAMES



DIAPHRAGM FOR STEEL FRAMES

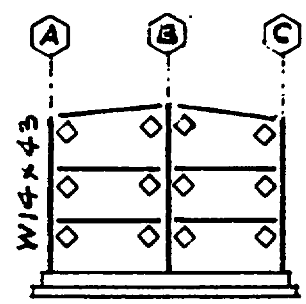


STEEL FRAMES AT (N)&(S) WALLS



VERTICAL LOAD + SEISMIC

LINES 2, 4, 6.



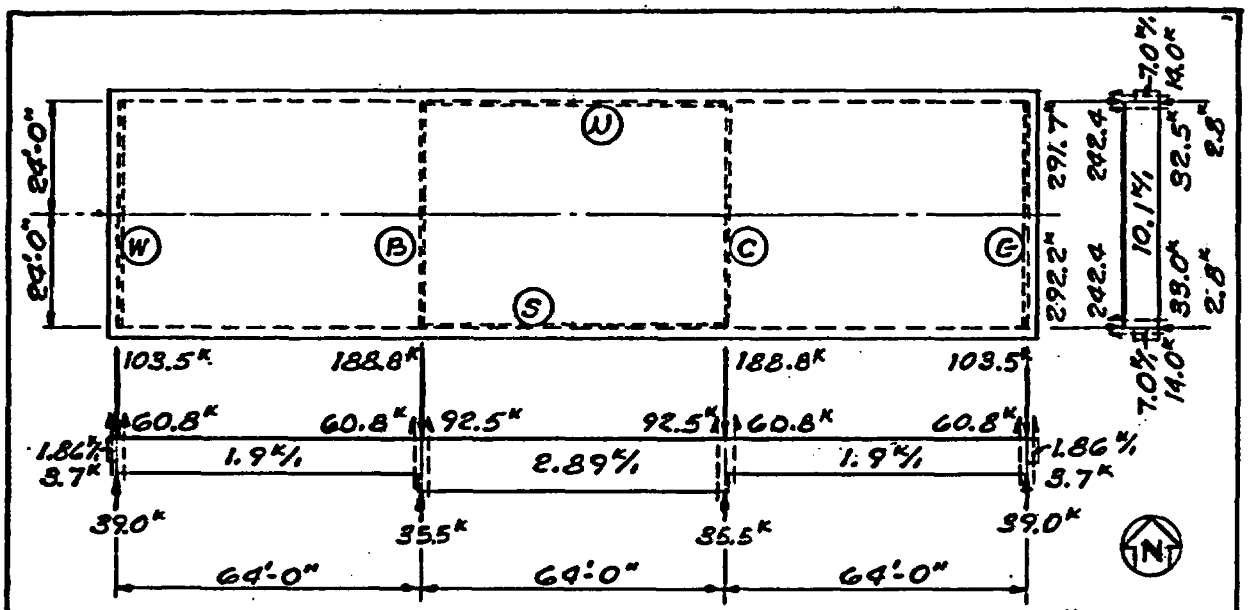
VERTICAL LOAD ONLY

LINES 1, 3, 5, 7.

TRANSVERSE STEEL FRAMES

◇ DENOTES FRAMING CONNECTION FOR SHEAR & CHORD FORCES; OTHER CONNECTIONS TO DEVELOP FLEXURAL CAPACITY FOR FRAME ACTION AS WELL AS SHEAR AND CHORD FORCES.

BEAMS IF EMBEDDED IN SHEAR WALLS SHALL BE DESIGNED TO CARRY THE WEIGHT OF CONCRETE IN THE STORY ABOVE.



TOTAL W_R 584K
LOADING @ 100% g

LOADING FOR ROOF DIAPHRAM

EXTERIOR WALLS (ACCOUNT FOR PERCENTAGE OF SOLID WALL-WINDOWS OUT)

METAL PANEL
WALL WT. 4 PSF × 5.5' = 22#/l × 62.8 = 1381.6# × 2 = 2,760#

10" CONC. WALLS { E & W WALLS: .833 × 6.5 × 150 = 813#/l
N & S WALLS: .833 × 5.5 × 150 = 687 }

W = 813 × 1.0 = 813 × 48 = 39,024# ← FRACTION SOLID

E = 813 × 1.0 = 813/1626#/l × 48 = 39,024#

N = 687 × .75 = 522 × 63.2 = 32,548#

S = 687 × .76 = 522/1037#/l × 63.2 = 32,900#

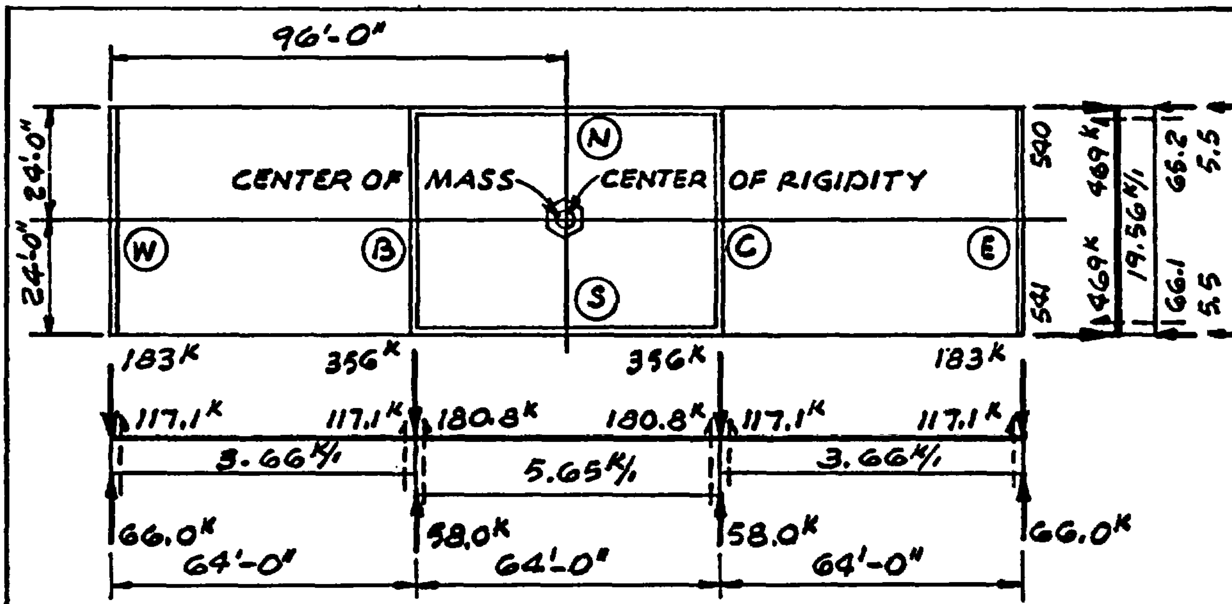
B OR C = 813 × 91% SOLID = 740#/l × 48.00 LENGTH = 35,520#

E-W LOADS

ROOF = 35.7 × 196 = 6997#/l
WALLS E & W = 1626
WALLS B & C 2 × 740 = 1480
10,103#/l

N-S LOADS

ROOF = 35.7 × 52' =	1856	ROOF	1856
WALLS N & S	= 1037	EXT. WALL 2 × 22 =	44
CENTER BAY	= 2893#/l	END BAYS	= 1900#/l



TOTAL $W_3 = 1078^K$

LOADING @ 100% g

LOADING FOR 3RD FLOOR DIAPHRAGM (2ND FLOOR SAME)

EXTERIOR WALLS (ACCOUNT FOR PERCENTAGE OF SOLID WALL - WINDOW OUT)

METAL PANEL

WALL WT. 4 P.S.F. × 11' = 44 × 62.8' = 2.76^K × 2 = 5,520#

10" CONC. WALL = .833 × 11' × 150 = 1375#/

W = 1375 × 1.0 = 1375 × 48 = 66,000#

E = 1375 × 1.0 = $\frac{1375}{2750\#/}$ × 48 = 66,000#

N = 1375 × 0.75 = 1031 × 63.3 = 65,262#

S = 1375 × 0.75 = $\frac{1045}{2076\#/}$ × 63.3 = 66,149#

B OR C = 1375 × 0.91 = 1251#/ × 46.33 = 57,970#

N-S LOADS

CENTER BAY

FLOOR 74.5 × 48 = 3576
WALLS N & S = 2076
5652#/

END BAYS

FLOOR 74.5 × 48 = 3576
EXT. WALL 2 × 44 = 88
3664#/

E-W LOADS

FLOOR 74.5 × 192 = 14,304
WALLS E & W = 2750
WALLS B & C 2 × 1251 = 2,502
19,556#/

LATERAL FORCES

	h	D	$T = \frac{.05h}{\sqrt{D}}$	$C = \frac{1}{\sqrt{T}}$
N - S	33'	48'	0.238	0.137 } BUT NEED NOT
E - W	33'	192'	0.119	0.193 } EXCEED 0.12

ASSUME $S = 1.5$

$CS = 0.12 \times 1.5 = 0.18$ BUT NEED NOT TO EXCEED 0.14

$Z = 1 \quad I = 1.0 \quad K = 0.8$

$V = ZIKCSW = 1 \times 1.0 \times 0.8 \times 0.14W = 0.112W$ BOTH DIRECTIOS

LEVEL	h_x	Δh	W_x	$W_x h_x$	$\frac{wh}{\Sigma wh}$	F	V	ΔM_{OT}	M_{OT}
ROOF	33'	11	584^k	19,272	.35	107^k	107^k	1177	
3RD	22'	11	1078	23,716	.43	132	239^k	2629	1177
2ND	11'	11	1078	11,858	.22	68	307^k	3377	3806
			$W = 2740^K$	54,846	1.0	307			7183

$V = 0.112 \times 2740 = 307^k \quad F_T = 0$ SINCE $T < 0.7$ SEC.

STORY FORCES FOR DESIGN

LEVEL	SHEAR WALL: F_x		STEEL FRAME $0.25F_x$	
ROOF	107^k	DISTRIBUTE TO CONC. SHEAR WALLS IN PROPORTION TO THEIR RELATIVE RIGIDITIES. INCLUDE ACCIDENTAL TORSION. SIM. TO A-1	27^k	DISTRIBUTE TO STEEL FRAMES IN PROPORTION TO RELATIVE RIGIDITIES INCLUDE ACCIDENTAL TORSION. SIM. TO A-3
3RD	132^k		33^k	
2ND	68^k		17^k	
	307^k		77^k	

Example A-5 7 of 7 Steel Frames, Concrete Walls

DESIGN EXAMPLE: A-6

BUILDING WITH A WOOD BOX SYSTEM:

Description of Structure. A two-story wood framed classroom building in Zone 3, using wood floor and roof decks and wood stud walls. Girders and columns on centerline of building support roof rafters and floor joists. The structural concept is illustrated on Sheets 2 and 3.

Construction Outline.

Roof:
- Composition & gravel.
- 1" diagonal sheathing.
- Wood rafters, wood girders, and columns.
- Ceiling (drywall + acoustic tile).

2nd Floor:
- 3/4" plywood sheathing.
- Asphalt tile.
- Wood floor joists, steel girders & columns.
- Ceiling (drywall + acoustic tile).

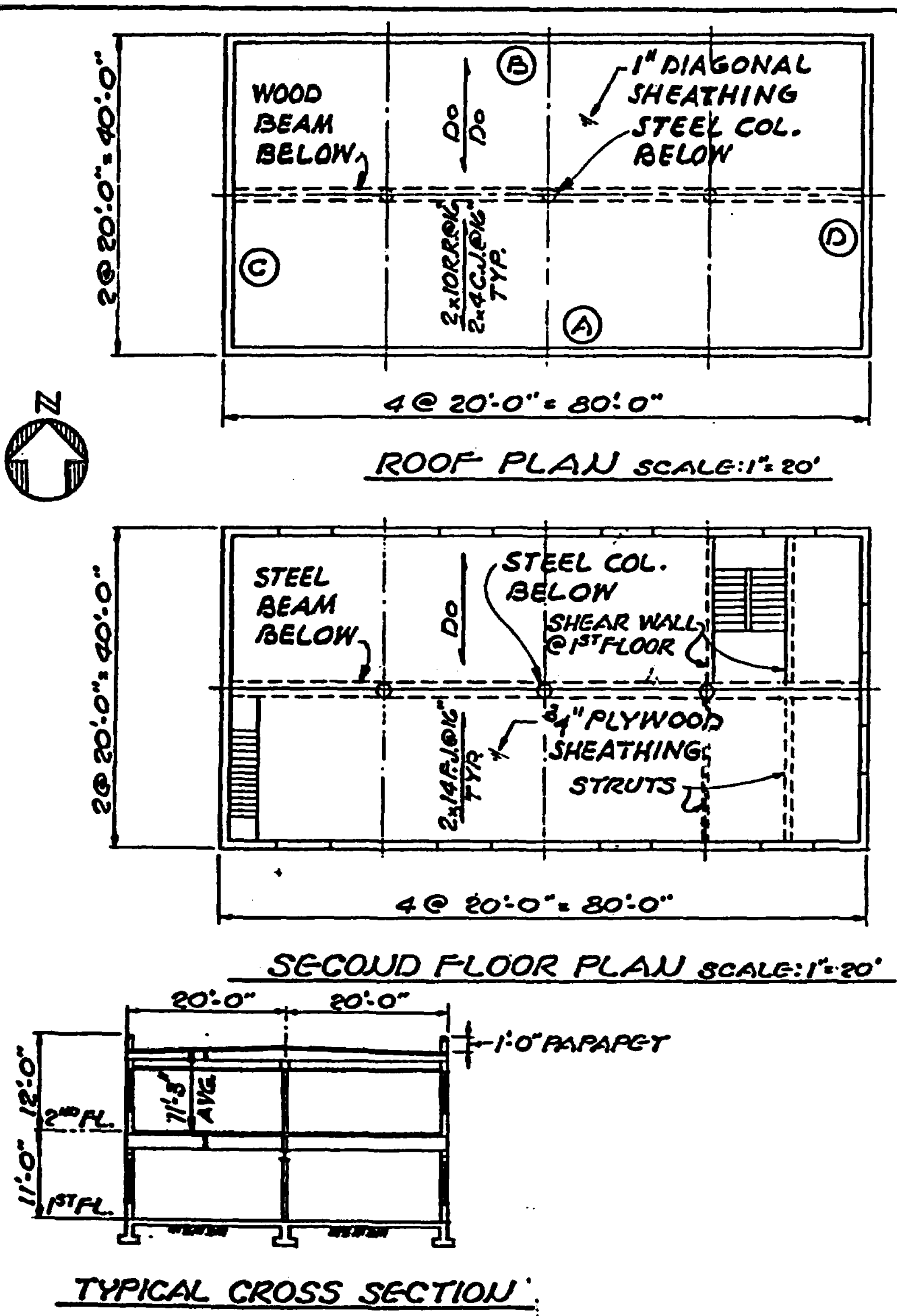
1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Wood stud bearing walls with exterior and interior plaster.

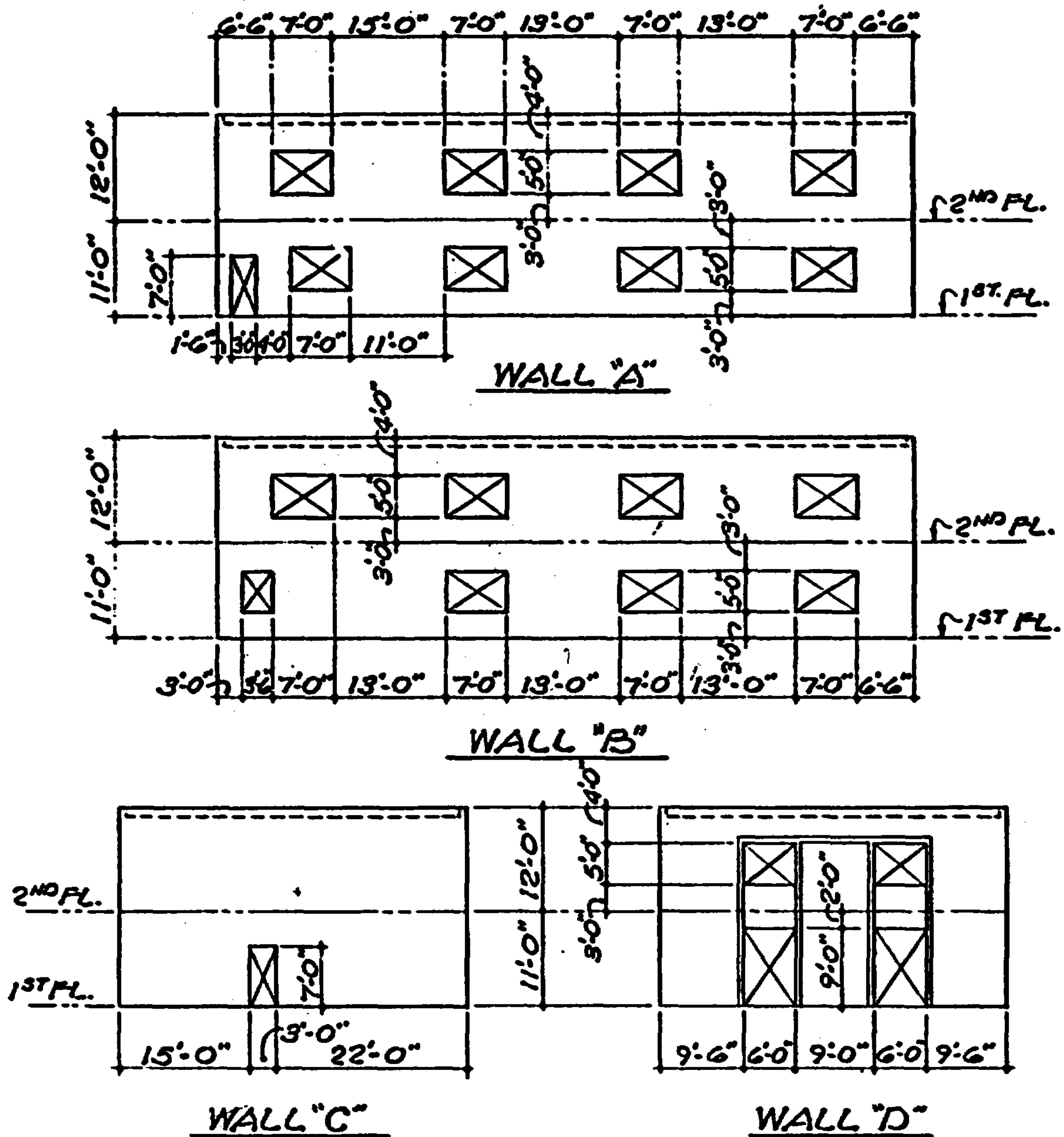
Partitions:
- The stair enclosure walls are wood stud with plywood sheathing on one. Other interior walls are removable drywall.

Design Concept. There is a line of columns and girders on the centerline of the building, but the exterior walls are bearing walls. Thus the structure does not have a complete vertical load-carrying space frame and is a Wood Box System with a K-factor of 1.00. The diagonally-sheathed roof acts as a diaphragm spanning between exterior walls. This is a very flexible diaphragm incapable of transferring significant rotational forces. The plywood sheathed second floor is a flexible diaphragm. This second floor diaphragm is interrupted by a stairwell. The permanent stair enclosure walls running in a north-south direction are therefore used as shear walls.

Discussion. The accompanying computations show the load diagrams and distribution of horizontal forces to the various shear walls and the unit shear and chord stresses in the diaphragm. Attention is called to the two second-floor struts which must transfer diaphragm shears to the shear walls on each side of the stairs. Double joists are used for these struts. Plywood sheathing is given for one of the stair walls. As this wall is short, it will be provided with special tie-down fastenings. Shear in piers of each wall are computed as proportional to the solid space between openings.



Example A-6 2 of 13 Wood Box



EXTERIOR WALL ELEVATIONS

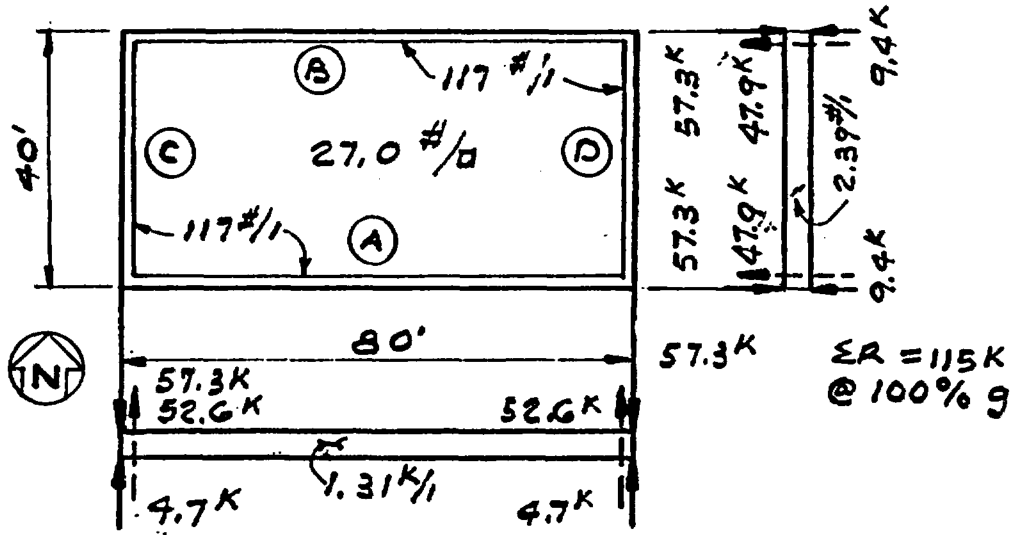
LOADS FOR ROOF DIAPHRAGM

ROOF

COMPO & GRAVEL ROOFING	=	6.0
1" DIAG. SHEATHING	=	1.5
RAFTERS & CEILING JOISTS	=	3.5
CEILING (DRYWALL + AC. TILE)	=	5.0
MISCELLANEOUS	=	1.0
DL	=	17.0
ADD PARTITIONS FOR SEISMIC		10.0
		27.0 PSF FOR SEISMIC

WALLS 11' HIGH & 1 FT. PARAPET,
STUDS & PLASTER 18 PSF × 6.5' = 117.0#/l

LOADING DIAGRAM - ROOF DIAPHRAGM



(N-S) LOADS

27#/□' × 40' = 1080#/l
117#/l × 2 = 234
1314
WALL C OR D 117 × 40 = 4680#

(E-W) LOADS

27 × 80 = 2160
117 × 2 = 234
2394
WALL A OR B 117 × 80 = 9360#

LOADS FOR 2ND FLOOR DIAPHRAGM

FLOOR

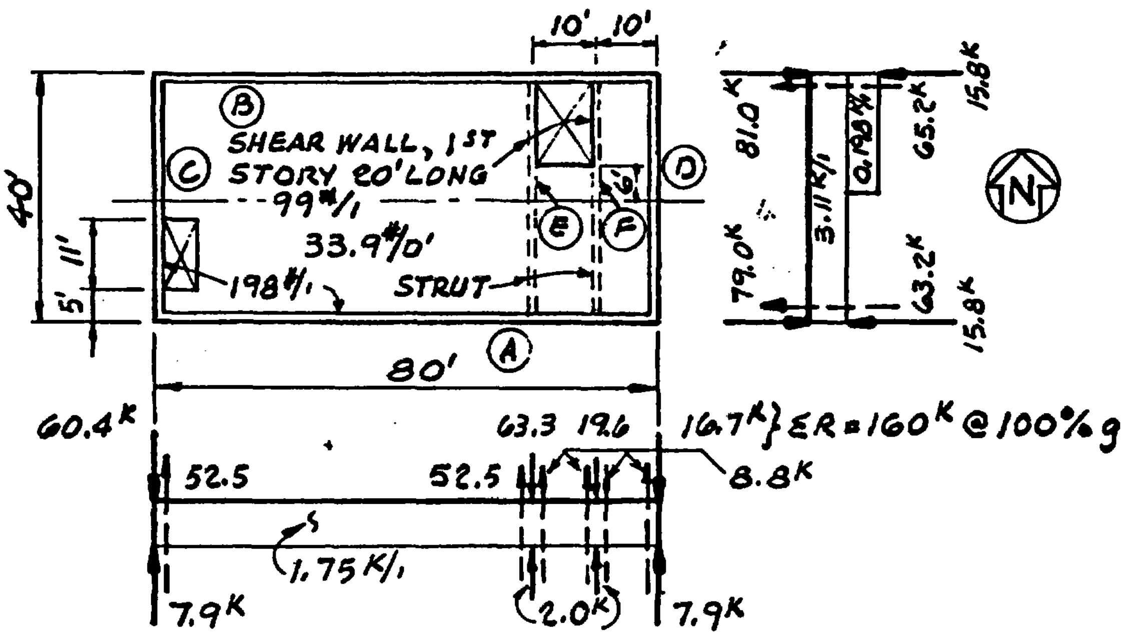
ASPHALT TILE	=	1.0 #/□'
3/4" PLYWOOD SHEATHING	=	2.3
FLOOR JOISTS	=	4.6
STEEL BEAMS & COLUMNS	=	1.0
CEILING	=	5.0
PARTITIONS	=	20.0
FLOOR DEAD LOAD	=	33.9 #/□'

WALLS

EXTERIOR: 18 #/□' × 11' = 198 #/'

INTERIOR: 18 × 5.5 = 99

LOADING DIAGRAM - 2ND FLOOR DIAPHRAGM



(N-S) LOADS

33.9 × 40 = 1356

198 × 2 = 396

1752 #/'

WALL C OR D = 198×40 = 7920#

WALL E OR F = 99×20 = 1980#

(E-W) LOADS

33.9 × 80 = 2712

198 × 2 = 396

3108

WALL E & F = 99×2 = 198#

WALL A OR B = 198×80 = 15,840#

LATERAL FORCES

$V = ZIKCSW$

ZONE 3: $Z = \frac{3}{4}$; $I = 1.0$; $K = 1.0$ (TABLE 3-3)

$T = 0.05h/\sqrt{D}$ $\quad h = 22.25'$ $\quad C = 1/15\sqrt{T}$

	D	T	C	
LONGIT.	80'	0.124	0.189	NEED NOT
TRANSV.	40	0.176 Sec.	0.159	EXCEED 0.12

$S = 1.5$, $CS = 0.12 \times 1.5 = 0.18$, BUT NEED NOT EXCEED 0.14

$V = \frac{3}{4} \times 1.0 \times 1.0 \times 0.14W = 0.105W$

STORY FORCE $F_x = \frac{wh}{\Sigma wh}(V)$

LEVEL	W_x	h	wh	$\frac{wh}{\Sigma wh}$	Fx
ROOF	115^K	22.25'	2559	0.59	17.0
2^{ND} FLR.	160^K	11.0'	1760	0.41	11.8
W = 275			4319	1.0	28.8

$V = 0.105 \times 275^k = 28.8^k$

ROOF DIAPHRAGM

LATERAL FORCE STORY FORCE = 17.0^k

DIAPHRAGM FORCE (3-3(J)2d)

$F_{px} = \frac{\Sigma F_i}{\Sigma W_i} W_{px} = \frac{17}{115} W_{px} = 0.148\, W_{px}$ ← GOVERNS

MIN. $F_{px} = 0.14\,ZIW_{px} = 0.14 \times 3/4 \times 1 \times W_{px} = 0.105\, W_{px}$

MAX. $F_{px} = 0.30\,ZIW_{px} = 0.30 \times 3/4 \times 1 \times W_{px} = 0.225\, W_{px}$

DIAPHRAGM STRESSES $w = 0.148\, W_{px}$

	BENDING M $= wL^2/8$	CHORD FORCE $= M/D$	SHEAR V $= wL/2$	$v = V/D$
N-S	$(1.31 \times 0.148) \times 80^2/8 = 155^k$	$\div 40' = 3.88^k$	7.76^k	$\div 40' = 0.19^k/_{FT}$
E-W	$(2.39 \times 0.148) \times 40^2/8 = 71^k$	$\div 80' = 0.89^k$	7.07^k	$\div 80' = 0.09^k/_{FT}$

SHEATHING $v_{MAX.} = 190^{\#}/_{1}$

1x DIAGONAL SHEATHING - DOUGLAS FIR

VERY FLEXIBLE DIAPHRAGM: WEB: F = 250 (TABLE 5-1 & 5-5)
ALLOWED DIAPHRAGM LENGTH = 2 × WIDTH (TABLE 5-1)
ALLOWED SHEAR = 300 #/FT. (TABLE 5-5)

CONNECTIONS

CHORD SPLICE NEAR MIDSPAN OVER WALL A OR B $P = 3880^{\#}$

TOP PLATE OF STUD WALL IS CHORD. LAP PLATES AND CONNECT WITH 3-3/4"φ BOLTS EACH SIDE OF SPLICE. CAPACITY IN SINGLE SHEAR IN 1½" MEMBERS = $3 \times 1350^{\#} \times 1.33 = 5.40^k$.

CHORD SPLICE NEAR MIDSPAN OVER WALL C OR D $P = 890^{\#}$

EDGE ROOF RAFTER IS CHORD. PROVIDE 7-16d NAILS EACH SIDE OF SPLICE. CAPACITY = $7 \times 107^{\#} \times 1.33 = 996^{\#}$

DIAPHRAGM CONNECTION WALL A OR B $v = 90^{\#}/_{1}$

BLOCKING TO BLOCKING & BLOCKING TO PLATE (SECT. A, FIGURE 5-33). PROVIDE 2-16d (OR METAL FRAMING ANCHORS) BETWEEN RAFTERS. CAPACITY = $(2 \times 107 \times 1.33) \div 1.33^{FT} = 214^{\#}/_{FT}$

DIAPHRAGM CONNECTION TO WALL C OR D $v = 190^{\#}/_{1}$

RAFTER TO BLOCKING & BLOCKING TO TOP PLATE (SECT. C, FIGURE 5-33) USE 16d @ 18" O.C. CAPACITY = $(107^{\#} \times 1.33 \div 0.67 = 212^{\#}/_{FT}$

Example A-6 7 of 13 Wood Box

2ND FLOOR DIAPHRAGM

LATERAL FORCE   STORY FORCE = 11.8^K $\frac{11.8}{160} = 0.0738$

DIAPHRAGM FORCE

$F_{px} = \frac{28.8}{275} W_{px} = 0.105\ W_{px}$ ←— GOVERNS

MIN. $F_{px} = 0.105\ W_{px}$

DIAPHRAGM STRESSES

	BENDING M $WL^2/8$	CHORD FORCE $= M/D$	SHEAR V $= WL/2$
N-S	$(1.75 \times 0.105) \times \overline{60}^2/8 = 82.7^{K'}$	$\div 40' = 2.07^K$	5.52^K
E-W	$(3.12 \times 0.105)\ \overline{40}^2/8 = 65.5$	$\div 80' = 0.82^K$	6.55^K

WALL	L	V	v	CASE *	BOUNDARY NAILS	PANEL NAILS	ALLOWED v
A	80'	6550#	82#/'	3	6" C.C.	6" C.C	215#/'
B, WEST OF STAIR	60'	"	109				
C, NORTH OF STAIR	24'	5520'	230	1	6" C.C.	6" C.C.	285#/'
E, PLUS STRUT	40'	"	138				

* SEE TABLE 5-6:
UNBLOCKED DIAPHRAGM, C-C EXT-APA PLYWOOD.
USE VALUES FOR 5/8" PLYWOOD, 10d NAILS, 2× MEMBERS.

FLEXIBILITY

USE L = 60' (DIAPH. SPANING FROM WALL C TO E)

FORMULA 5-33: $q_{AVE} = 230$ $q_d = 285$

$$F = \frac{33{,}000\, q_{AVE}}{q_d^{\,2}} = \frac{33{,}000 \times 230}{(285)^2} = 93$$

TABLE 5-1: DIAPH. IS "FLEXIBLE". MAX SPAN = 100' > 60' OK WITH FLEXIBLE WALLS AND NO CALCULATED TORSION IN THE DIAPH. THE MAX. SPAN = 3 × DEPTH OR 3×40' = 120' > 60' OK

2ND FLOOR DIAPHRAGM - CONT'D.

CONNECTIONS

CHORD SPLICE AT WALL A OR B $P = 2070^{\#}$

SIMILAR TO ROOF. USE 2-$\frac{5}{8}$"ϕ BOLTS EACH SIDE.

CAPACITY $= 2 \times 1000^{\#} \times 1.33 = 2660^{\#}$

CHORD SPLICE AT WALL C OR D $P = 820^{\#}$

SIMILAR TO ROOF. USE 6-16d NAILS EACH SIDE.

CAPACITY $= 6 \times 107^{\#} \times 1.33 = 854^{\#}$

DIAPHRAGM AT WALL A

WALL ABOVE, SOLE PLATE TO BLOCKING, 2-16d BETWEEN RAFTERS FOR $90^{\#}/FT$, AS AT TOP PLATE.

BLOCKING TO BLOCKING AND BLOCKING TO TOP PLATE:

SHEAR FROM ROOF & FLOOR $= 90 + 82 = 172^{\#}/FT$

USE 2-16d BETWEEN RAFTERS, SIMILAR TO ROOF.

CAPACITY $= 214^{\#}/FT$.

DIAPHRAGM AT WALL B

SIMILAR TO WALL A. SHEAR $= 90 + 109 = 199^{\#}/FT$, USE 2-16d

DIAPHRAM AT WALL C

WALL ABOVE, SOLE PLATE TO EDGE RAFTER.

USE 16d @ 8" C.C. FOR $190^{\#}/FT$, AS AT ROOF.

RAFTER TO BLOCKING AND BLOCKING TO TOP PLATE:

SHEAR FROM ROOF & FLOOR $= 190 + 230 = 420^{\#}/FT$

USE 16d @ 4" C.C.

CAPACITY $= 107^{\#} \times 1.33 \div 0.33' = 431^{\#}/FT$

DIAPHRAGM AT WALL E

NO WALL ABOVE.

STRUT IS DOUBLE JOIST EXTENDING OVER WALL E, SIMILAR TO PLAN A, FIG. 5-34.

$$\text{STRUT FORCE} = \left(\frac{20'}{40'} \times 0.183^{K}/FT \times \frac{60'}{2}\right) + \left(\frac{20'}{26'} \times 0.183 \times \frac{10'}{2}\right)$$

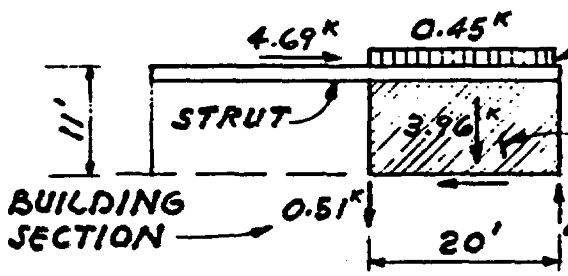
$$= 3.45^{K}$$

USE 2-$\frac{3}{4}$"ϕ BOLTS, DOUBLE JOIST TO $\angle 4 \times 4 \times \frac{1}{4}$ AND 2-$\frac{3}{4}$"ϕ BOLTS, ANGLE TO DOUBLE TOP PLATE OF WALL E.

CAPACITY IN SINGLE SHEAR IN 3" OF WOOD WITH METAL SIDE PLATE $= 2 \times (1.25 \times 1470^{\#}) \times 1.33 = 4888^{\#}$

SHEAR WALLS

WALL E 2ND STORY FORCE = 0.074 x FLOOR WEIGHT



2ND FLR. DL = $\frac{1.33'}{2}$ x 20' x 33.9 PSF
= 450#

WT. OF WALL = 20' x 11' x 18 PSF
= 3960#

TOTAL DL = 4410#

SEISMIC LOADS (0.074 x FLOOR WT.)
DIAPH. LOADING DIAGRAM

FROM THE WEST	0.074 x 52.5K	= 3.89K
FROM THE EAST	x 8.8'	= 0.66
FROM THE WALL	x 2.0	= 0.15
	WALL V	= 4.69K

WALL SHEAR $V = \frac{4690\#}{20'} = 235\#/'$

USE 5/16" STRUCT. I EXT.-APA PLYWOOD ONE SIDE 6d @ 4" AT PANEL EDGES, 6d @ 12" AT INTERMEDIATE SUPP'TS. ALLOWABLE SHEAR = 300#/' (FIGURE 6-15)

OVERTURNING M = 4690# x 11' = 51,600#'

WALL REACTIONS = $\frac{4410\#}{2} \pm \frac{51,600\#'}{19'}$ = 2205 ± 2716

$= \begin{cases} 4921 \text{ DOWN} \\ 511 \text{ UPLIFT} \end{cases}$

TIE DOWN (FIG. 6-16)

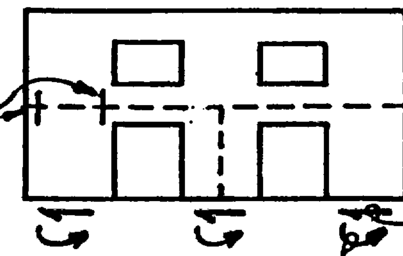
POST BOLTS TO ANGLE: 2-5/8" φ SINGLE SHEAR 2½" NET, WITH METAL SIDE PLATES: ALLOW 2 x (1.25 x 1020#) x 1.33 = 3392# > 511#. ANCHOR BOLT: 5/8" φ ALLOW 1.33 x 3000 = 4000#. ∠4 x 4 x 5/8 x 0'-3½": S = (3½ - 7/8")(5/8")²/6 = .173 IN³
M = 511# x (2½ - 1) = 767#" F = 767/.173 = 4888 PSI

SHEAR WALLS CONT'D.

WALL (D)

TREAT AS 3 EQUAL CANTILEVER PIERS
ASSUME NO MOMENT DEVELOPED IN SPANDRELS.

SPLICE FOR VERTICAL CONTINUITY TYPICAL AT ALL PIERS.



$\rightarrow .148 \times 57.3^K = 8.49^K \times 22.3 = 189$

$\rightarrow .074 \times 16.7^K = 1.24^K \times 11.0 = 14$

$\overline{9.72^K}$ $\quad M_{OT}\ \overline{203^{K'}}$

$V = \frac{9.72}{3} = 3.24^K$

$M = \frac{203}{3} = 67.8^{K'}$

WALL SHEAR
1ST. STORY:

$v = \frac{9720}{3 \times 9} = 360\ ^{\#}/_{FT}$

EXT. LATH & PLASTER	-200
INT. " " "	-100
1" DIAG. SHEATING	---300
ALLOW. v	$= 600^{\#}/_{FT}$

WALL HT/WIDTH

$= \frac{11.3}{9} = 1.25 < 2$ OK

OVERTURNING:	2ND STORY	1ST STORY
WT. OF WALL	$9' \times 12' \times 18\ psf = 1944^{\#}$	$9' \times 23' \times 18 = 3613^{\#}$
DL OF ROOF	$15' \times \frac{8}{12} \times 17 psf = 170$	170
DL OF FLOOR		$15 \times \frac{8}{12} \times 34 = 340$
TOTAL DL	$\overline{2114^{\#}}$	$\overline{4123^{\#}}$
O.T. MOMENT	$\frac{8.48^K \times 11.3}{3} = 31.9^{K'}/PIER$	$67.8^{K'}/PIER$
O.T. FORCE	$\frac{31.9}{8.5'} = 3.75^K$	$\frac{67.8}{8.5'} = 7.98^K$
REACTIONS	$\frac{2114}{2} \pm 3750$	$\frac{4123}{2} \pm 7980$
	$= \begin{cases} 4807 & \text{DOWN} \\ 2963 & \text{UPLIFT} \end{cases}$	$= \begin{cases} 10673 & \text{DOWN} \\ 5918 & \text{UPLIFT} \end{cases}$

Example A-6 11 of 13 Wood Box

SHEAR WALLS- CONT'D

WALL (D) CONT'D

UPLIFT AT 2ND FLOOR F = 2963#

PROVIDE VERTICAL CONTINUITY WITH METAL SPLICE PLATE 3/16" x 2½" WITH 2-5/8" BOLTS EACH END. ALLOW $2 \times (1.25 \times \frac{2030}{2}) \times 1.33 = 3375^{\#} > 2963$

TIE DOWNS AT 1ST FLOOR F = 5918#

USE STIFFENED ANGLE (FIG. 6-16) 3-3/4"φ BOLTS TO 4x4 POST, SINGLE SHEAR IN 2½" NET WITH METAL SIDE PLATE. ALLOW $3 \times (1.25 \times \frac{2870^{\#}}{2}) \times 1.33 = 7157^{\#}$

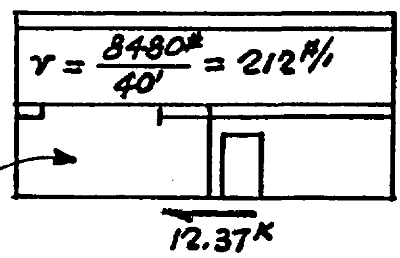
7/8"φ ANCHOR BOLTS WITH 3"x3" WASHER

NET AREA OF WASHER = 8 IN.^{2}
ALLOW 600 PSI IN B'R'G: 600 PSI x 8 x 1.33 = 6400#

NOTE THAT FOOTING MUST BE REINFORCED SO THAT THE BOLT CAN PICK UP ABOUT 1.5 CU. YDS OF CONCRETE.

SHEAR WALLS - CONT'D

WALL (C)



$\to .148 \times 57.3^K = 8.48^K \times 22.3 = 189^K$

$\to .074 \times 52.5^K = 3.89^K \times 11.0 = 43$

$12.37^K \quad M_{OT} = 232^{K'}$

$v = \frac{12,370}{15' + 22'} = 335^{\#/}$

WT. OF WALL $= 18\,psf \times 23$
$= 414^{\#/}$

RESISTING MOMENT

$M_R = 0.414^{K/} \times 40' \times 20'$
$= 331 > 232^{K'}$
$\therefore$ NO UPLIFT

SHEATHING:

2ND STORY:
EXT. + INT. LATH & PLASTER
$200 + 100 = 300^{\#/} > 212$

1ST STORY:
ADD 1" DIAG. SHEATHING
$300 + 300 = 600^{\#/} > 335$

WALL (A) ((B) SIMILAR)

	F	V	NET L	v
ROOF $0.148 \times 57.3 =$	8.48	8.48^K	52'	$163^{\#/}$
FLOOR $0.074 \times 79.0 =$	5.85	14.33	47.5'	301

SHEATHING:

2ND STORY EXT. + INT. L + P $300^{\#/} > 163$

1ST STORY ADD 4 LET-IN BRACES

$$300^{\#/} + \frac{4 \times 1000^{\#}}{47.5} = 384^{\#/} > 301$$

DESIGN EXAMPLE: A-7

SPECIAL CONFIGURATION:

Description of Structure. A one-story industrial garage building in Seismic Zone 3. The north, east, and west walls are concrete bearing walls. The south wall is largely open for drive-in access and has concrete columns and concrete beams over the openings. The roof is concrete slab and beams. The structural concept is illustrated on Sheets 2 and 3.

Design Concept. The roof is a reinforced concrete beam and slab system forming a relatively rigid diaphragm, even with a 6 to 1 length-width ratio. The north, east, and west walls are concrete bearing walls. The south wall is a rigid frame. The lateral forces are resisted by shear walls. The building is a Box System with a K-factor of 1.33.

Discussion. An estimate of the relative deflections and stiffnesses of the north wall versus the south wall rigid frame indicates that practically all of the east-west forces would be carried by the north wall. The resulting rotation is resisted by the east and west walls. A computation of the deflection of the roof diaphragm in resisting north-south forces is shown. The transverse bents formed by the south wall columns, the transverse roof beams, and a portion of the north wall are checked to see if these bents are adequate for the vertical load carrying capacity and the induced moment due to 3/K times the deflection resulting from the lateral forces. The vertical load stresses in the south wall beams will be combined with chord stresses of the roof diaphragm.

LATERAL FORCES $\qquad V = ZIKCSW$

ZONE 3 , $Z = \frac{3}{4}$; $I = 1.0$, $K = 1.33$

$T = 0.05h/\sqrt{D}$, $h = 16$, $C = \frac{1}{15\sqrt{T}}$

	D	T	C	
LONGIT.	24'	0.163 sec.	0.167	BUT NEED NOT
TRANSV.	144'	0.0667	0.258	EXCEED 0.12

ASSUME GEOTECHNICAL INVESTIGATION SHOWS THAT $T_S > 2.5$ SEC.

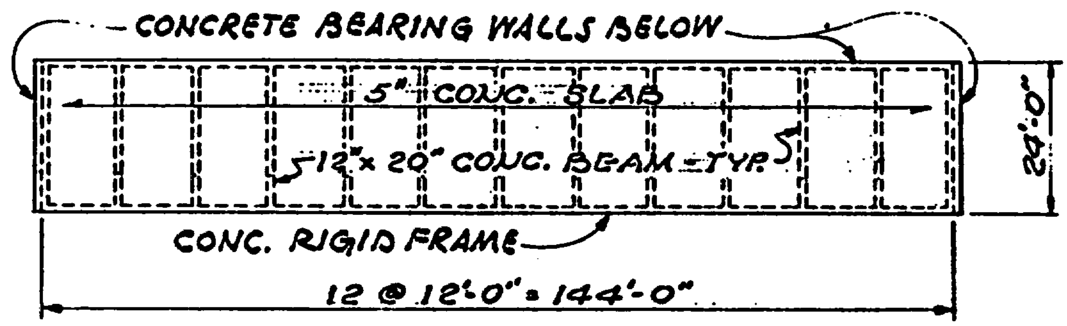
$T_{MIN} = 0.3 \quad T/T_S = 0.3/2.5 = 0.12$

$S = 1.0 + T/T_S - 0.5\,(T/T_S)^2$ FOR $T/T_S \leq 1.0$

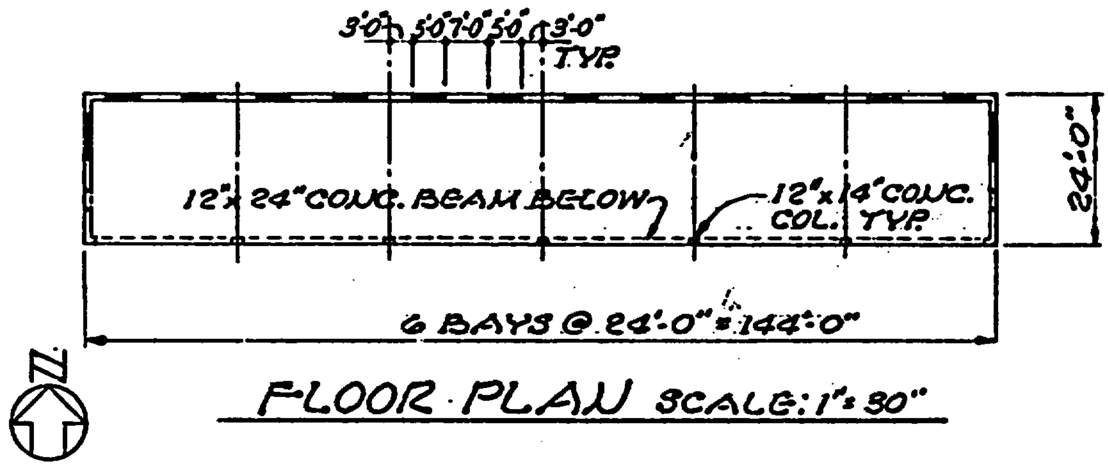
$S = 1.0 + 0.12 - 0.5\,(0.12)^2 = 1.11$

$C_S = 0.12 \times 1.11 = 0.133$

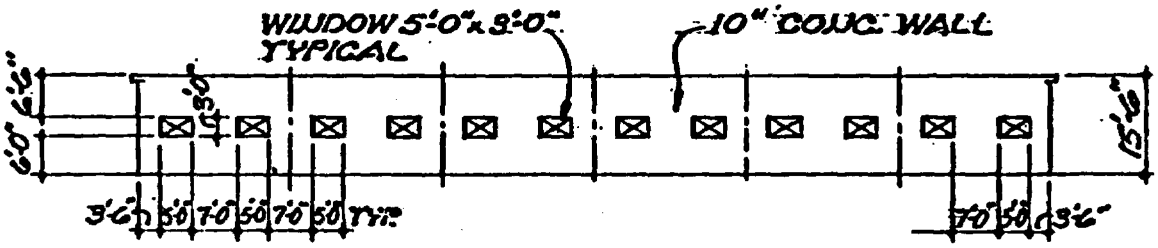
$V = \frac{3}{4} \times 1.0 \times 1.33 \times 0.133\,W = 0.133\,W$



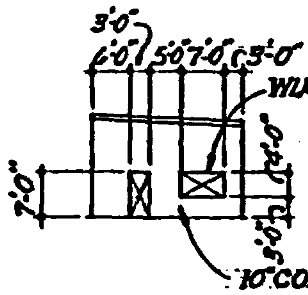
ROOF PLAN SCALE: 1" = 30'



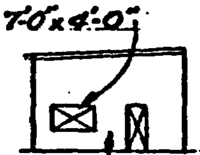
FLOOR PLAN SCALE: 1" = 30"



NORTH ELEVATION SCALE 1"=30'

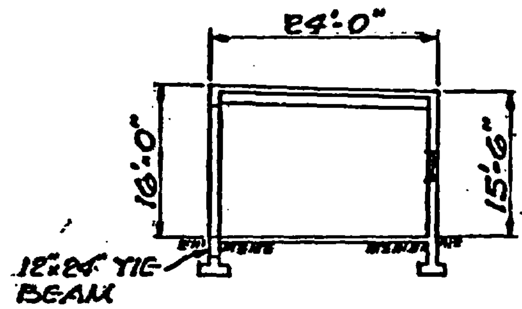


EAST ELEVATION

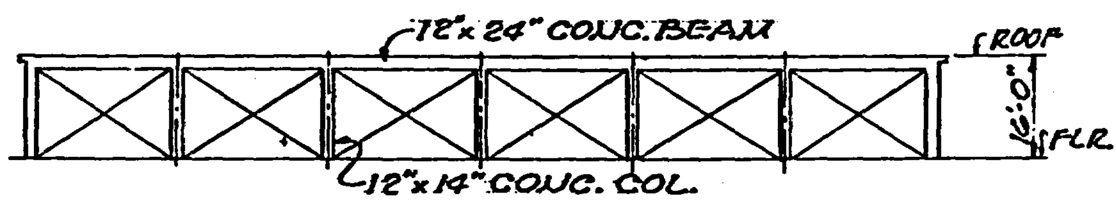


WEST ELEVATION

SCALE 1"=30'



TYPICAL CROSS SECTION SCALE 1"=20'



SOUTH ELEVATION SCALE: 1"=30"

Example A-7 | 3 of 9 | Special Configuration

ROOF D.L.

COMPO & GRAVEL ROOF 7.0
5" CONC. SLAB 63.0
BEAMS 16.0
86.0 #/□'

12"x 24" CONC. BEAM = 150 x 1.58 = 237 #/'
COLUMNS = 1 x 1.17 x 150 x 14/2 = 1228 #

EXTERIOR WALLS

N. WALL 10" CONC. 125 x 7.55 = 945 #/'
E. & W. END WALLS 125 x 7.88 = 985 #/'

SEISMIC N-S

ROOF 86 x 24 = 2060
N. WALL = 945
BEAM = 237
COLS. = 5 x 1228 / 144 = 43
DOOR OR COVER 10x7 = 70
.133 x 3355 = 446 #/'

END WALL 985 x 24 = 23,600 x .133 = 3.1 K

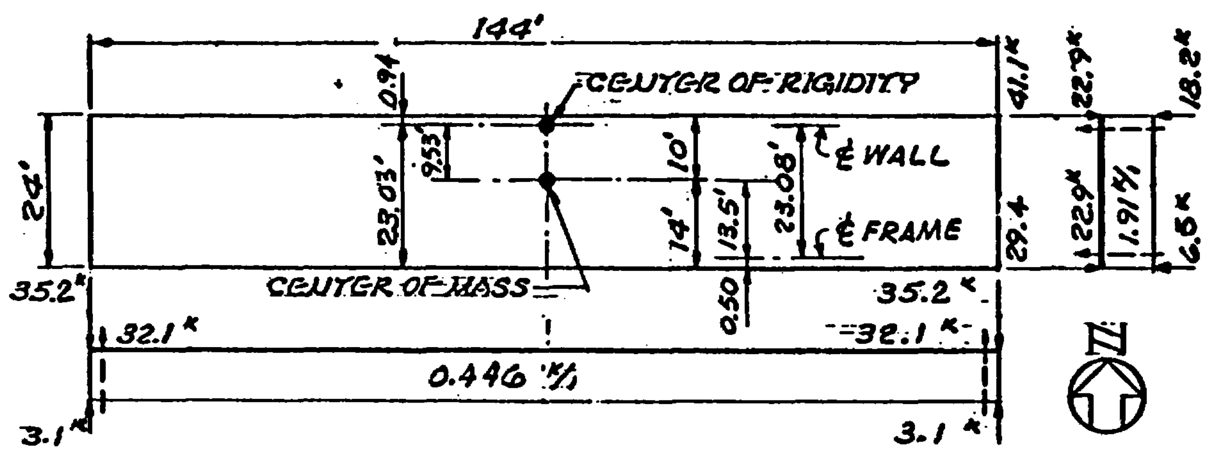
SUMMARY

2 END WALLS @ 3.15 = 6.3
DIAPHRAGM 0.45 x 144 = 64.8
TOTAL SEISMIC WEIGHT = 71.1 K

SEISMIC E-W

ROOF 86 x 144 = 12,400
WALLS 2 x 985 = 1,970
.133 x 14,370 = 1910 #/'

N. WALL 945 x 144 = 136,000 x .133 = 18.2 K
S. WALL
BEAM 237 x 144 = 34,200
COLS. 1228 x 5 = 6,140
COVER 60 x 144 = 8,650
.133 x 48,990 = 6.5 K



SEISMIC LOADS

RELATIVE RIGIDITIES

NORTH WALL

USING CHART FOR DEFLECTION (FIG. G-11) DEFLECTIONS TABULATED BELOW FOR 10" WALLS, ARE 12/10 TIMES THE CHART VALUE WHICH ARE FOR 12" WALLS.

PIER	h	d	h/d	Δ	K
1^{CF}	3	3.5	.86	.0852	11.75
2^{RF} TO 12 INCL.	3	7.0	.43	.0458	21.82 x 11 = 240.0
13^{CF}	3	3.5	.86	.0852	11.75
					263.5
$WALL^{RC}$	15.5	144	.1077	.0108	
$BAND^{RF}$ @ WIND.	3	144	.0208	.0024	

$\Delta PIERS(1\text{-}13) = \frac{1}{263.5} = 0.0038$

$\Delta WALL = .0108 - .0024 + 0.0038 = 0.0122$

$K(WALL) = \frac{1}{.0122} = 82.6$

NOTE:
CF INDICATES CORNER PIER, FIXED CONDITION
RC INDICATES RECTANGULAR PIER CANTILEVER CONDITION

SOUTH WALL (RIGID FRAME)

DEFLECTION OF PIERS - BEAM FIXED

PIER	h	d	h/d	Δ	K
1^{CF} & 7^{CF}	14	1.17	12	32.8	.0305 x 2 = .061
$2\text{-}6^{RF}$ INCL.	14	1.17	12	49	.0204 x 5 = .1020
$WALL^{RC}$	16	144	.111	.0095	
@OPNGS. RC	14	144	.097	.0085	

$\Sigma\Delta = 12 \times .0694 + 12^3 \times .0185 = 32.8$

$\Sigma\Delta = 12 \times .08334 + 12^3 \times .0278 = 49.0$

$\Delta 1\text{-}7 = \frac{1}{.061 + .1020} = 6.14$

$\Delta WALL = .0095 - .0085 + 6.14 = 6.141$

DEFLECTION DUE TO ROTATION OF BEAM

$\theta = \frac{M_o}{EI}\left(a - \frac{a^2}{L} - \frac{L}{3}\right)$ @ CENTER OF BEAM $a = \frac{L}{2}$ *

$\frac{M_o}{EI}\left(\frac{-L}{12}\right)$ USE $P = 1000^{K}$

$M = 1.000 \times 14 = 14.000^{K'}$

$\theta = \frac{14{,}000{,}000 \times 12}{8{,}000{,}000 \times 18824}\left(\frac{-24' \times 12''/1}{12}\right) = 0.0972$

$\theta h = 0.0972 \times 14 \times 12 = 16.33''$

TOTAL DEFLECTION $= 6.14 + 16.33 = 22.47'$

$K(WALL) = \frac{1}{22.47} = 0.0445$

* FORMULA FROM ROARK, FORMULAS FOR STRESS AND STRAIN

RELATIVE RIGIDITIES (CONT.)

EAST & WEST WALLS.

PIER	h	d	h/d	Δ	K
1^{CF}	7	6	1.17	.132	7.6
2^{RF}	4	5	.8	.0966	10.35
3^{CF}	4	3	1.33	.168	5.96
4_a^{CF}	4	15	.267	.0222	
4^{CF}	7	15	.466	.0408	
5_a^{CF}	7	24	.292	.0246	
5^{CC}	15.75	24	.656	.0804	

$\Delta_{2\text{-}3} = \dfrac{1}{10.35+5.96} = 0.0613$

$\Delta_{1\text{-}4} = \dfrac{1}{7.6+12.5} = .0498$

$\Delta_4 = .0408 - .0222 + .0613 = .0799 \quad K_4 = \dfrac{1}{.0799} = 12.5$

$\Delta_{WALL} = .0804 - .0246 + .0498 = .1056$ (FOR $P = 1{,}000^k$)

$K_{WALL} = \dfrac{1}{.1056} = 9.47 \quad \Delta$ WALL FOR $P = 35.6^k = \dfrac{35.6}{1000}(.1056)$

$= 0.0038''$

CENTER OF RIGIDITY

N-S: ON BLDG. ℄ BY INSPECTION

E-W: NORTH WALL $K = 82.6 \times 23.08 = 1906 \quad \bar{X} = \dfrac{1906}{82.64} = 23.06$

SOUTH WALL $K = \underline{0.0445} \times 0 = \underline{0}$

$82.64 \qquad 1906$

AT N. WALL

CENTER OF MASS

N-S: ON BLDG. ℄ BY INSPECTION

E-W: NORTH WALL $R = 41.1 \times 23.08 = 948.6 \quad \bar{X} = \dfrac{948.6}{70.5} = 13.5'$

SOUTH WALL $R = \underline{29.4} \times 0 = \underline{0}$

$70.5^k \qquad 948.6$

DISTRIBUTION OF FORCES

ACCIDENTAL $e = 0.05(144) = 7.2'$

E-W: $V = 41.1 + 29.4 = 70.5^K \quad e = 23.06 - 13.5 = 9.56'$

$M_T = 70.5^K \times 9.56' = 674^K$

WALL	K	d^2	Kd^2	$\frac{M}{\Sigma Kd^2}$	÷d	(E-W) S	(N-S) S	K LOAD	ACCID. TORS.	DESIGN LOAD	FACTORED LOAD 1.4	FACTORED LOAD 2.0
N	82.6	0	0	0.0069 (E-W) 0 (N-S)	0	0	0	70.5	0	70.5	98.6	141
S	.04	532.7	21.3		-23.08	∓0.01	0	0.03	∓0.020	0.05	0.07	0.10
	82.64											
E	9.47	5130	48,581		+71.58	±4.68	0	35.2	±3.54	39.9	55.9	79.8
W	9.47	5130	48,581		-71.58	∓4.68	0	35.2	∓3.54	39.9	55.9	79.8
	18.94		97,184									

E-W: $\dfrac{M}{\Sigma Kd^2} = \dfrac{674}{97{,}184} = 0.0069$

DIRECT SHEAR $= (K/\Sigma K)V$

TORS. SHEAR $= Kd(M_T/\Sigma Kd^2)$

E·W EARTHQUAKE

NORTH WALL

FACTORED DESIGN LOAD = 2 × 70.4 = 140.8^K

$$v_u = \frac{V_u}{\varphi A_c} = \frac{140{,}800}{0.85 \times (144' - 60')12''/_1 \times 10''} = 16 \text{ PSI}$$

SOUTH WALL

RIGIDITY ANALYSIS FINDS NEGLIGIBLE DESIGN FORCE FOR THE SOUTH WALL.
∴ DESIGN THE FRAME FOR VERTICAL LOAD PLUS INDUCED MOMENTS DUE TO 3/K TIMES THE DISTORTION RESULTING FROM THE LATERAL FORCE.

$$\Delta = \frac{3}{1.33}(\Delta_{N.WALL} + \Delta_{DIAPH.})$$

IN THE CASE WHERE THE DIAPHRAGM FLEXIBILITY WOULD PERMIT THE FRAME TO DEFLECT SIGNIFICANTLY MORE THAN THE NORTH SHEAR WALL, A DUCTILE FRAME WOULD BE PROVIDED.

N-S EARTHQUAKE

DIAPHRAGM $M = \frac{0.45 \times 144^2}{8} = 1166^{K'}$

CHORD $F = \frac{1166^{K'}}{23'} = 50.7^K$; $A_s = \frac{50.7 \times 1.4}{40 \times 0.9} = 1.97^{\square''}$

CONT. CHORD BARS 2-#9

END WALLS: DESIGN FORCE, F = 39.2K
FACTORED DESIGN FORCE FOR OVERTURNING = 1.4 × 39.2 = 54.9K; M_{OT} = 54.9 × 15.8' = 867K'
FACTORED DESIGN FORCE FOR SHEAR = 2.0 × 39.2 = 78.4 K

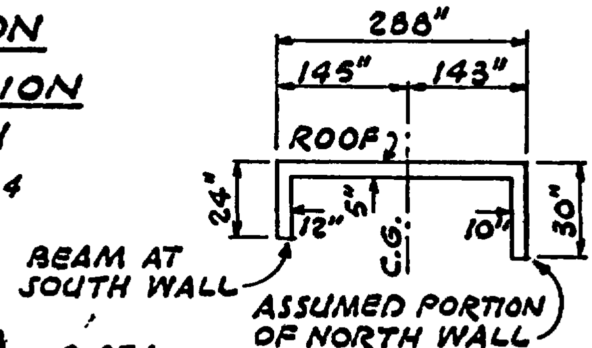
TRANSVERSE FRAMES

THESE WERE NEGLECTED IN THE RIGIDITY ANALYSIS. CHECK THAT THEY CAN TAKE 3/K TIMES THE DEFLECTION CALCULATED FOR THE ROOF DIAPHRAGM ACTING WITHOUT THE FRAMES.

DIAPHRAGM DEFLECTION

FLEXURAL DEFLECTION

ASSUMED SECTION



$I = 19{,}120{,}000\ IN^4$

$$\Delta_f = \frac{5wL^4 \times 1728}{384EI}$$

$$\Delta_f = \frac{5 \times 450 \times 143.17^4 \times 1728}{384 \times 3 \times 10^6 \times 19.12 \times 10^6} = 0.074$$

SHEARING DEFLECTION OF WEB

$$\Delta_w = \frac{AV.\ SHEAR\ PER\ FT. \times L \times f}{10^6} \qquad V_{AV} = \frac{32.4}{2 \times 24} = 675\ K/I$$

$$F = \frac{10^6}{8.5 \times 5 \times 150^{1.5}\sqrt{3000}} = 0.234$$

$$\Delta_w = \frac{675 \times 72 \times .234}{10^6} = 0.0114''$$

TOTAL DEFLECTION OF DIAPHRAGM BETWEEN END WALLS

$$\Delta_D = \Delta_F + \Delta_w = 0.074 + 0.0114 = 0.085\ IN$$

DEFLECTION OF END WALL = 0.0037

DEFLECTION OF FRAME BEAM WITH RESPECT TO GROUND

$$\Delta_B = 0.085 + 0.0037 = 0.089$$

REQUIRED FRAME DEFLECTION

$$\Delta = (3/K)\Delta_D = (3/1.33) \times 0.089 = 0.200''$$

TRANSVERSE FRAMES (CONT.)

STIFFNESS OF FRAME

$I_{AB} = \dfrac{14 \times 12^3}{12} = 2020\ IN^4$

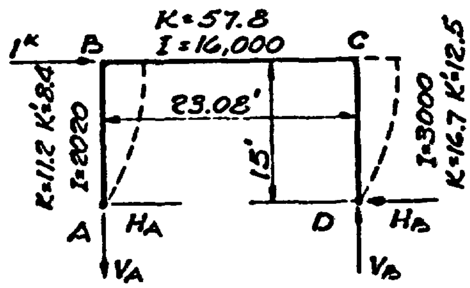
$K = \dfrac{2020}{15 \times 12} = 11.2$ FOR T-BEAM

$I_{BC} = \dfrac{2 \times 12 \times 20^3}{12} = 16{,}000\ IN^4$

$K = \dfrac{16{,}000}{23.08 \times 12} = 57.8$

$I_{CD} = \dfrac{36 \times 10^3}{12} = 3000\ IN^4$

$K = \dfrac{3000}{15 \times 12} = 16.7$



DEFLECTION OF FRAME FOR 1^K LOAD. FIXED JOINTS @ B & C
FIXED END MOMENTS OF COLUMNS $= -\dfrac{3EK\Delta}{L}$

JOINT	B		C	
	BA	BC	CB	CD
D.F.	.127	.873	.822	.178
F.E.M.	-.187			-.278
	+.024	+.163 +.114	+.229 +.081	+.049
	-.014	-.100 -.033	-.067 -.050	-.014
	+.004	+.029 +.020	+.041 +.014	+.009
	-.003	-.017 -.005	-.011 -.008	-.003
	-.001	+.004	+.007	+.001
	$-.175E\Delta$			$-.236E\Delta$

FIXED-END MOMENTS:

$M^F_{BA} = \dfrac{-3E\,11.2\,\Delta}{15 \times 12} = 0.187E\Delta$

$M^F_{CD} = \dfrac{-3E\,16.7\,\Delta}{15 \times 12} = 0.278E\Delta$

FINAL MOMENTS FOR $P = 1^K$:

$M_{BA} = 0.175E\Delta = 0.175 \times 438$
$= 76.7$ K-IN $(6.39K')$

$M_{CD} = 0.236 \times 438$
$= 103$ K-IN $(8.61K')$

Σ SHEARS $= 1^K - \dfrac{.175E\Delta}{15 \times 12} - \dfrac{.236E\Delta}{15 \times 12} = 0 \longrightarrow E\Delta = 438$

$\Delta = \dfrac{438}{3 \times 10^3} = 0.146''$

FRAME DEFORMATION COMPATIBILITY

FOR REQ'D FRAME DEFLECTION OF 0.200",

$M_{BA} = \dfrac{6.39K'}{0.146\ IN.} \times 0.200 = 8.8K'$

$M_{CD} = \dfrac{8.61}{0.146} \times 0.200 = 11.8K'$

WHEN COMBINED WITH GRAVITY LOADS THE RESULTING STRESSES ARE WITHIN THE ELASTIC LIMIT; P-Δ IS SMALL. ∴ OK.

DESIGN EXAMPLE: A-8

L-SHAPED BUILDING WITH A BOX SYSTEM:

Description of Structure. A three-story L-shaped Administration Building in Zone 3 with bearing walls in concrete, using a series of interior vertical load-carrying column and girder bents. The structural concept is illustrated on Sheets 2, 3, and 4.

Construction Outline.

Roof:
- Built-up, 5 ply.
- Metal decking with insulation board.
- Suspended ceiling.

2nd & 3rd Floors:
- Metal decking with concrete fill.
- Asphalt tile.
- Suspended ceiling.

1st Floor:
- Concrete slab-on-grade.

Exterior Walls:
- Bearing walls in concrete furred with GWB finish.

Partitions:
- Non-structural removable drywall.

Design Concept. Since the structure is without a complete load-carrying space frame, the K-factor is 1.33. The metal deck roof system forms a flexible diaphragm, therefore the roof loads are distributed to the shear walls by tributary area rather than by third story wall stiffnesses. The roof diaphragm, being flexible, will not transmit accidental torsion to the shear walls. The metal deck with concrete fill system for the floors form rigid diaphragms. The walls act as a series of vertical cantilever beams connected together by struts at the floor lines. The wall analysis utilizes the Design Curve for Masonry and Concrete Shear Walls on Figure 6.11.

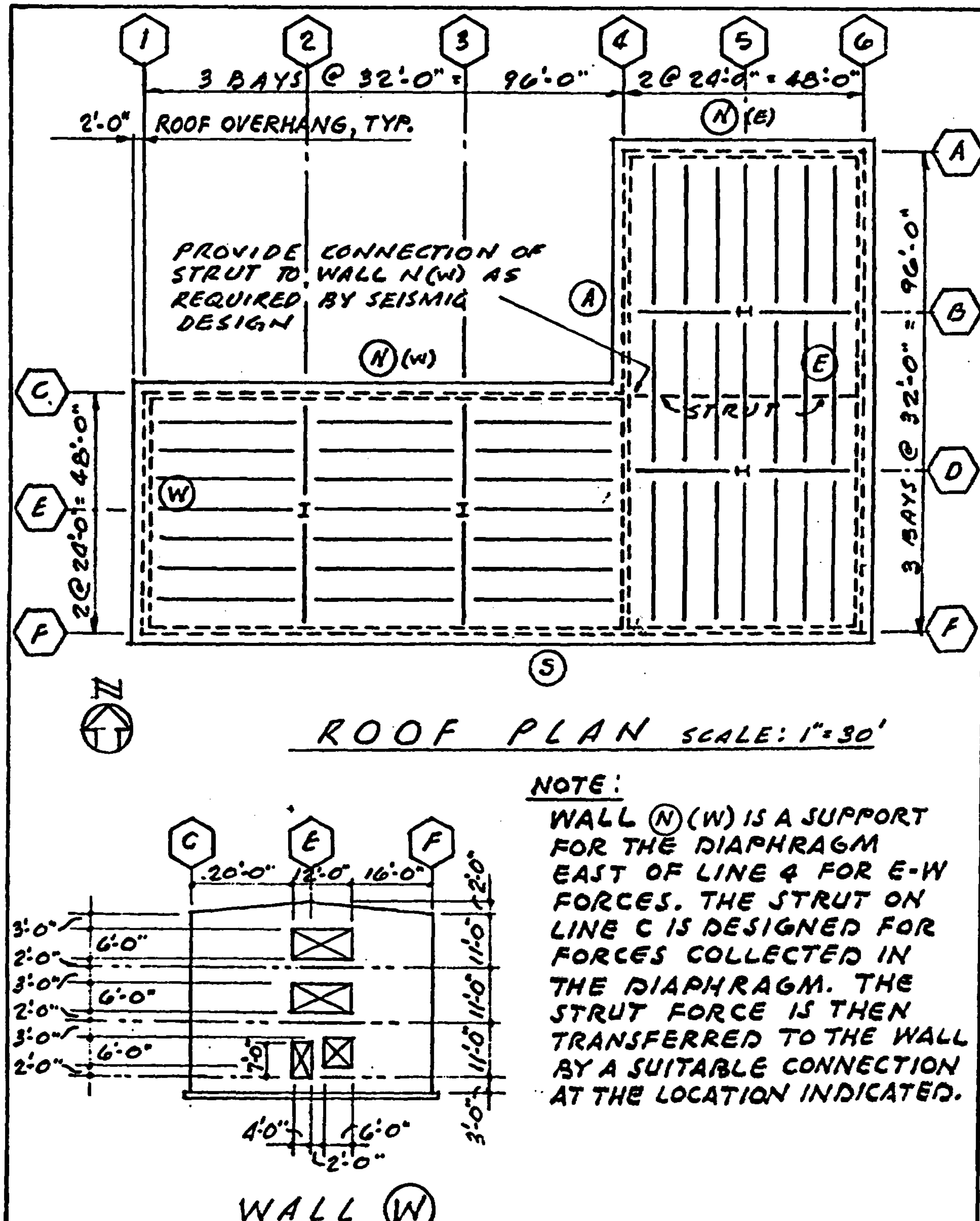
Loads.

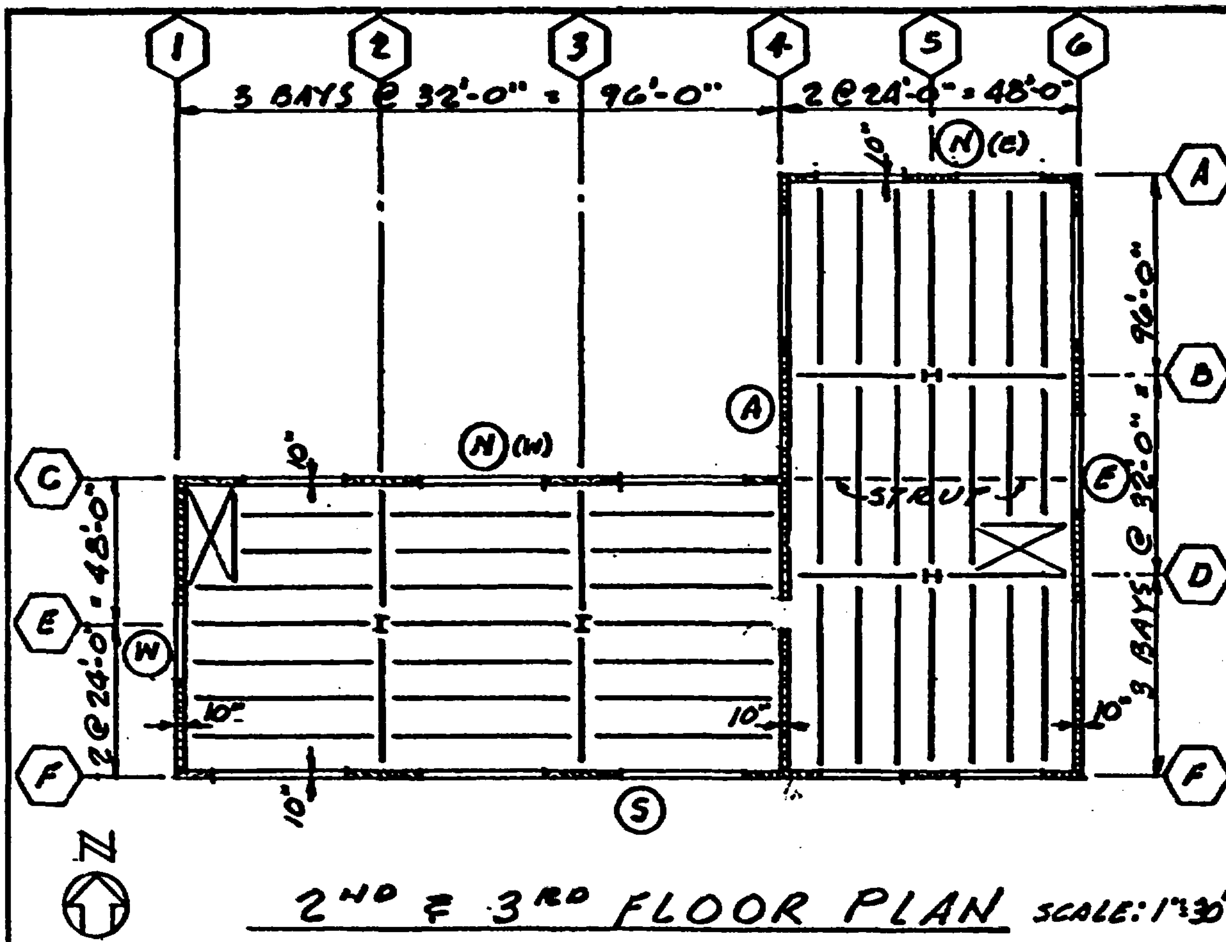
Roof:

5-ply roofing	=	6.0 p.s.f.
1" insulation	=	1.5
Steel decks	=	2.3
Steel purlins	=	3.7
Steel girders and columns	=	1.2
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load	=	25.7 p.s.f.
Add for seismic: Partitions		10.0
Total for seismic	=	35.7 p.s.f.

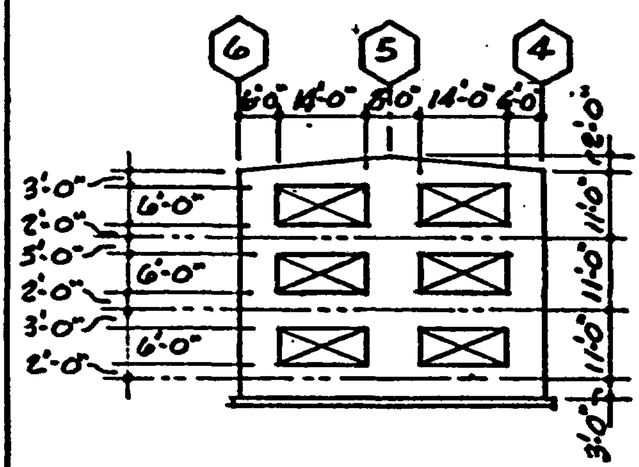
2nd & 3rd Floors:

Finish	=	1.0 p.s.f.
Steel deck	=	3.1
Concrete fill	=	32.0
Steel beams	=	5.9
Steel girders and columns	=	1.5
Partitions	=	20.0
Ceiling	=	10.0
Miscellaneous	=	1.0
Dead Load	=	74.5 p.s.f.
Live Load	=	50.0 p.s.f.

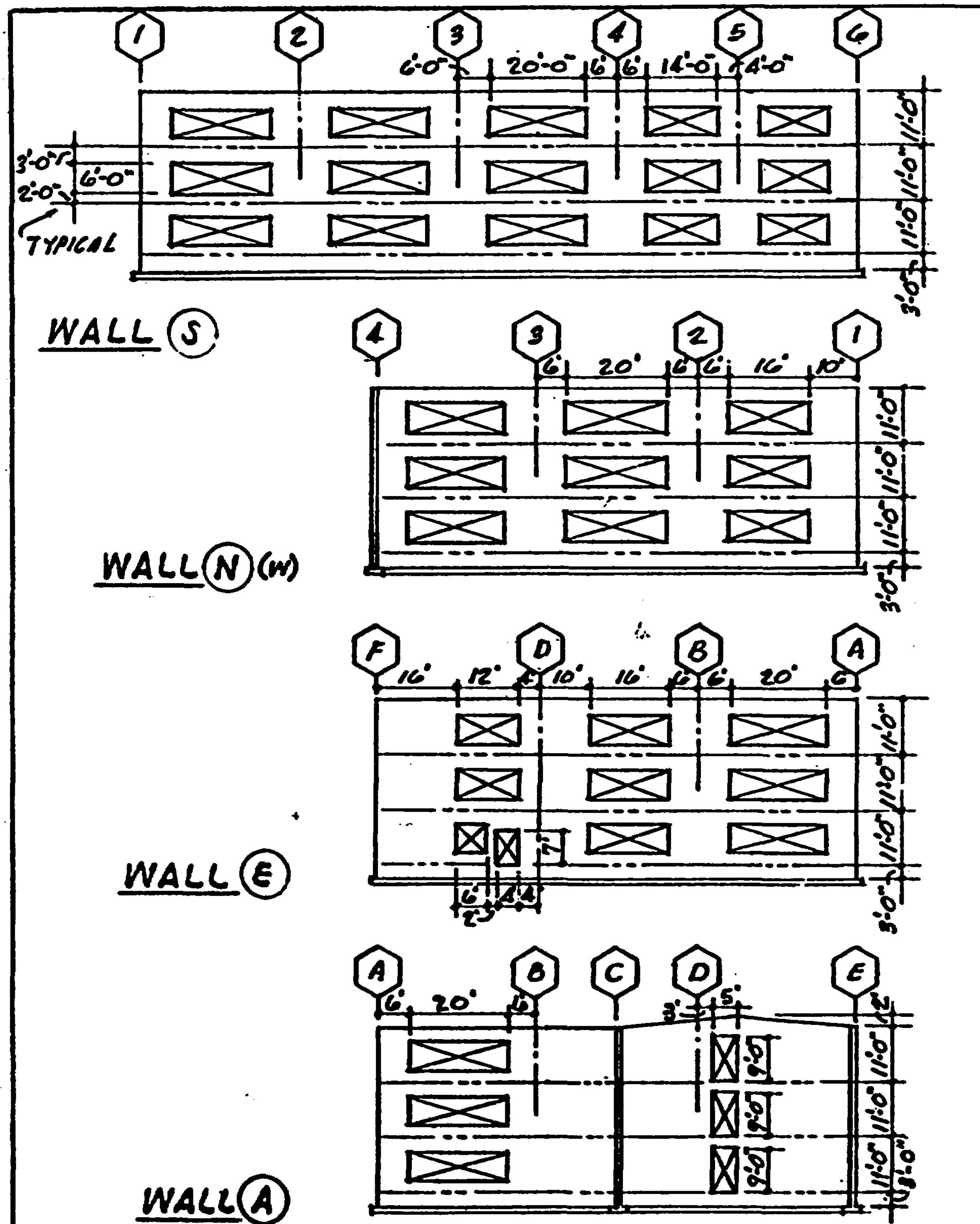

3 BAYS @ 32'-0" = 96'-0"
2 @ 24'-0" = 48'-0"
2'-0" ROOF OVERHANG, TYP.
N (E)
PROVIDE CONNECTION OF STRUT TO WALL N(W) AS REQUIRED BY SEISMIC DESIGN
A
N (W)
E
STRUT
W
2@20'-0" = 40'-0"
3 BAYS @ 32'-0" = 96'-0"
S
N
ROOF PLAN SCALE: 1" = 30'
NOTE:
WALL N (W) IS A SUPPORT FOR THE DIAPHRAGM EAST OF LINE 4 FOR E-W FORCES. THE STRUT ON LINE C IS DESIGNED FOR FORCES COLLECTED IN THE DIAPHRAGM. THE STRUT FORCE IS THEN TRANSFERRED TO THE WALL BY A SUITABLE CONNECTION AT THE LOCATION INDICATED.
20'-0"
12'-0"
16'-0"
3'-0"
2'-0"
6'-0"
2'-0"
11'-0"
7'-0"
4'-0"
6'-0"
2'-0"
WALL W



NOTE:
PROVIDE STRUT AND CONNECTION TO WALL ON LINE C SIMILAR TO ROOF.



WALL N (E)



Example A-8 4 of 8 L-Shaped Building

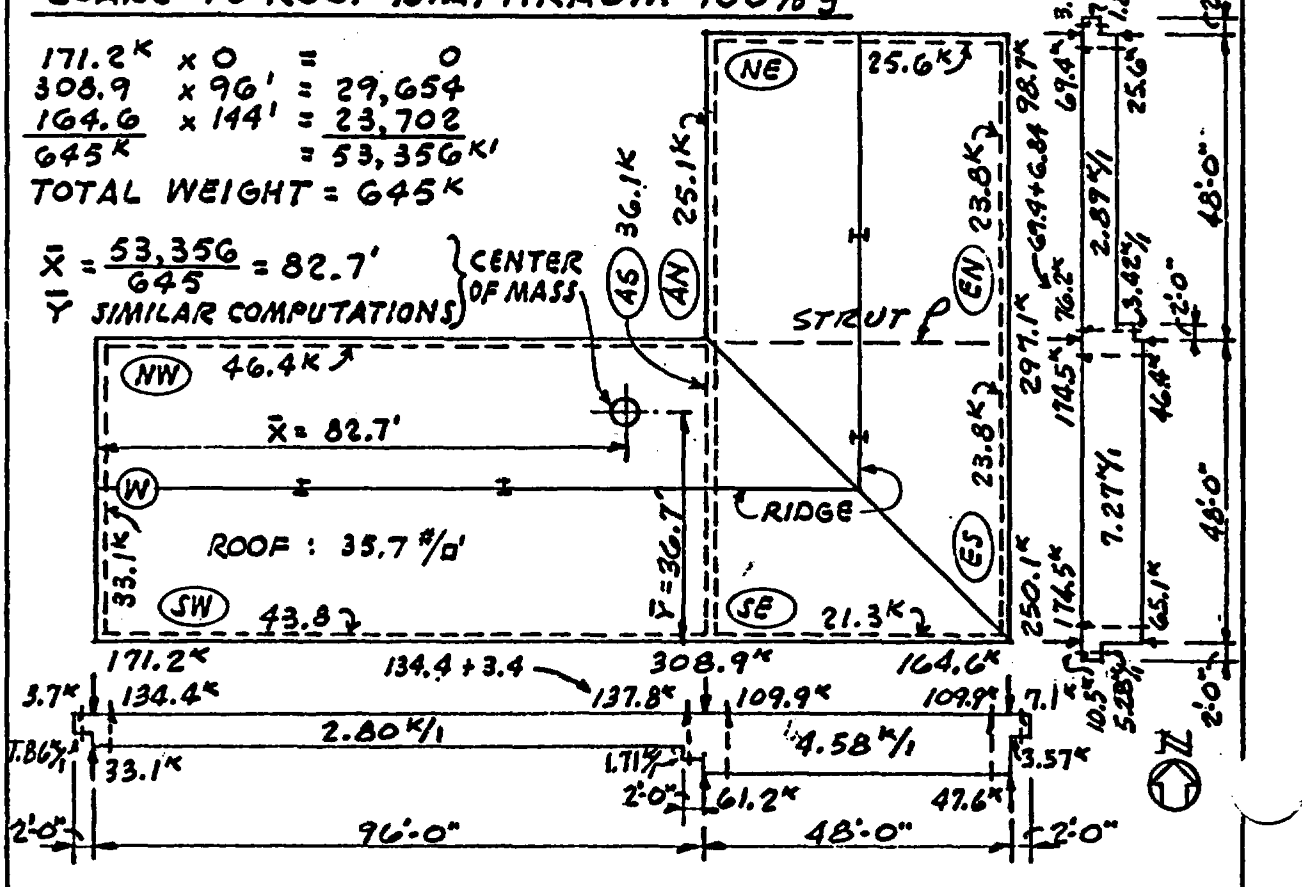
LOADS TO ROOF DIAPHRAGM 100% g

$$
\begin{array}{rcl}
171.2^K \times 0 & = & 0 \\
308.9 \times 96' & = & 29{,}654 \\
164.6 \times 144' & = & 23{,}702 \\
\hline
645^K & = & 53{,}356^{K'}
\end{array}
$$

TOTAL WEIGHT = 645^K

$\bar{X} = \frac{53{,}356}{645} = 82.7'$ } CENTER OF MASS

$\bar{Y}$ SIMILAR COMPUTATIONS



WALL	SOLID WT. #/l	% TOTAL AREA	NET WT. #/l	L'	W K	N-S WEST	N-S EAST	E-W SOUTH	E-W NORTH
NW	687	.71	488	95.17	46.4	488			
NE	813	.68	553	46.33	25.6		553		
SW	687	.67	460	95.17	43.8	460			
SE	687	.67	460	46.33	21.3		460		
W	813	.88	715	46.33	33.1			715	
EN	687	.73	502	47.17	23.8				502
ES	687	.73	502	47.17	23.8			502	
AN	687	.77	529	47.17	25.1				529
AS	813	.94	764	47.17	36.1			764	
				WALLS #/l		948	1013	1981	1031
				ROOF WIDTH		52'	100'	148'	52'
				ROOF WEIGHT #/l		1856	3570	5284	1856
				TOTAL #/l		2804	4583	7265	2887
				K/l		2.80	4.58	7.27	2.89

10" WALL WT.

125 #/□ × 6.5' = 813 #/l *
125 × 5.5' = 687 #/l
*GREATER HT. TO RIDGE

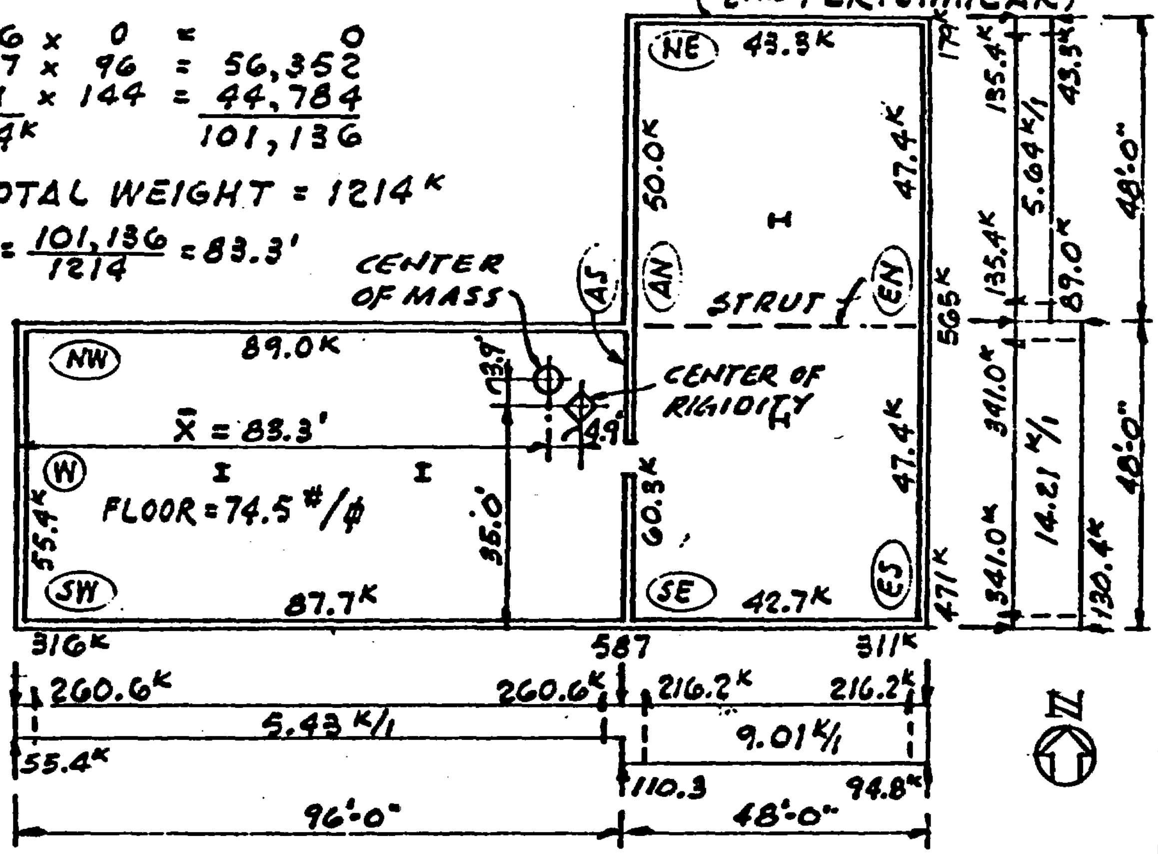
Example A-8 — 5 of 8 — L-Shaped Building

LOAD TO FLOOR DIAPHRAGM 100% 3RD FLR.
(2ND FLR. SIMILAR)

$$\begin{array}{rcrcr} 316 & \times & 0 & = & 0 \\ 587 & \times & 96 & = & 56,352 \\ 311 & \times & 144 & = & 44,784 \\ \hline 1214^K & & & & 101,136 \end{array}$$

TOTAL WEIGHT = 1214 K

$\bar{X} = \dfrac{101,136}{1214} = 83.3'$



WALL	SOLID WT. #/l	%TOTAL AREA	NET WT. #/l	L'	W^K	N-S WEST	N-S EAST	E-W SOUTH	E-W NORTH
NW	1375	.68	935	95.17	89.0	935			
NE	"	.68	935	46.33	43.3		935		
SW	"	.67	921	95.17	87.7	921			
SE	"	.67	921	46.33	42.7		921		
W	"	.87	1196	46.33	55.4			1196	
EN	"	.73	1004	47.17	47.4				1004
ES	"	.73	1004	47.17	47.4			1004	
AN	"	.77	1059	47.17	50.0				1059
AS	"	.93	1279	47.17	60.3			1279	
10" WALL WT.					WALLS #/l	1856	1856	3479	2063
125#/φ x 11' = 1375#/l					FLOOR WIDTH	48'	96'	144'	48'
					FLOOR WEIGHT	3578	7152	10728	3576
					TOTAL #/l	5434	9008	14,207	5639
					K/l	5.43	9.01	14.21	5.64

LATERAL LOADS

$V = ZIKCSW$

FOR LOCATION IN ZONE 3, $Z = 0.75$

$I = 1.0$, $K = 1.33$

TO OBTAIN C, CALCULATE $T = \frac{0.05h}{\sqrt{D}}$, AND $C = \frac{1}{15\sqrt{T}}$

FOR N-S DIRECTION

$T = \frac{0.05 \times 33}{\sqrt{96}} = 0.168$; $C = \frac{1}{15\sqrt{0.168}} = 0.163$

FOR E-W DIRECTION

$T = \frac{0.05 \times 33}{\sqrt{144}} = 0.138$; $C = \frac{1}{15\sqrt{0.138}} = 0.180$

BUT NEED NOT EXCEED 0.12

NO GEOTECHNICAL DATA IS AVAILABLE FOR THE SITE

∴ USE $S = 1.5$; $CS = 0.12 \times 1.5 = 0.18$

BUT CS NEED NOT EXCEED 0.14 IN BOTH DIRECTIONS

$V = \underset{Z}{0.75} \times \underset{I}{1.0} \times \underset{K}{1.33} \times \underset{CS}{0.14}W = 0.14W = 0.14 \times 3073 = 430^K$

BOTH DIRECTIONS

VERTICAL DISTRIBUTION OF LATERAL FORCES AND OVERTURNING MOMENTS

$F_x = \frac{(V - F_t)\omega_x h_x}{\Sigma \omega_i h_i}$; SINCE $T < 0.7$ SEC., $F_t = 0$

$F_x = V \frac{\omega_x h_x}{\Sigma \omega_i h_i}$

	h_x	Δh	ω_x	$\omega_x h_x$	$\frac{\omega_x h_x}{\Sigma \omega_x h_x}$	F_x	V_Y	$V_x h = \Delta M_x$	M_V
ROOF	33	11	645	21285	.347	149	149	1639	1639
3RD FLR.	22	11	1214	26708	.435	187	336	3696	5335
2ND FLR.	11	11	1214	13354	.218	94	430	4730	
TOTAL	-	-	3073	61347	1.000	430^K			10065

Example A-8 7 of 8 L-Shaped Building

ROOF DIAPHRAGM

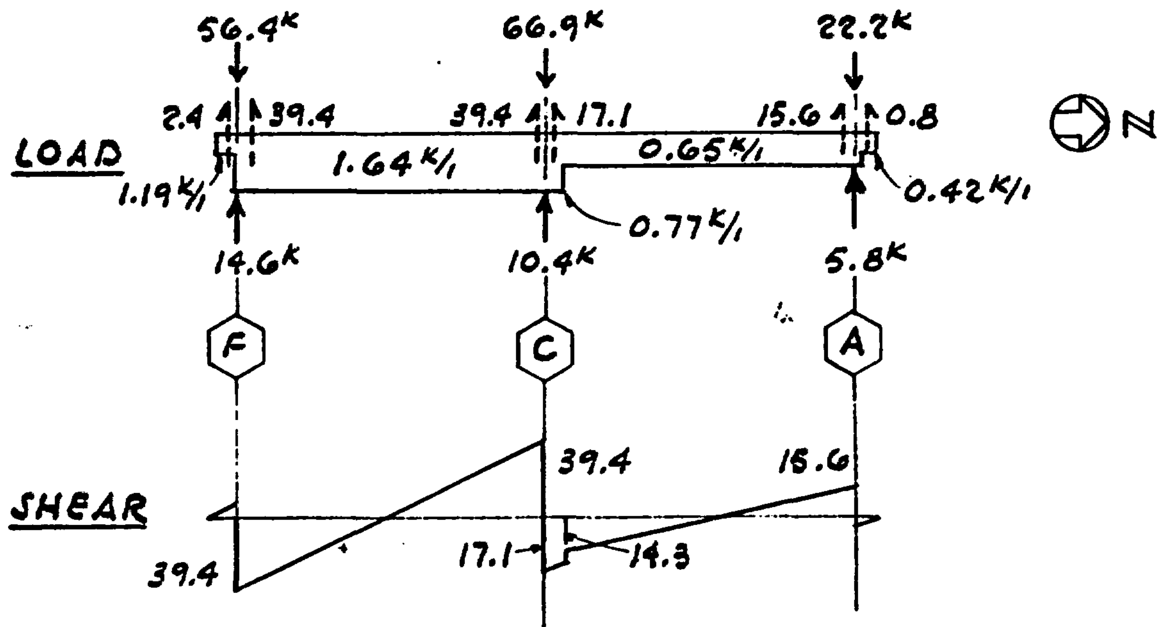
STORY FORCE = 149.5^K $\frac{F}{W} = \frac{149.5}{645} = 0.232$

DIAPH. FORCE PER EQN. 3-9:

$$F_{px} = \left(\frac{\Sigma F_i}{\Sigma w_i}\right) w_{px} = \frac{149.5}{645} w_{px} = 0.232\, w_{px}$$

$$\text{MAX } F_{px} = 0.3\,ZI\,w_{px} = (0.3 \times \tfrac{3}{4} \times 1.0)\,w_{px} = 0.225\, w_{px} \leftarrow \text{GOVERNS}$$

EAST-WEST EQ. MULT. LOAD DIAG., p. 5, BY 0.225
$\Sigma R = 145.5^K$



STRUT COLLECTS SHEAR FORCE BETWEEN LINES 4 & 6:

NORTH OF STRUT = $V = 17.1 - 2(0.77) = 15.6$
SOUTH OF STRUT = $V = \frac{48'}{144'} \times 39.4 = 13.1$
$\Big\}\ 28.7^K$

THE STRUT IS IN TENSION FOR EASTWARD FORCES, COMPRESSION FOR WESTWARD. SUITABLE CONNECTIONS MUST BE PROVIDED ACROSS EACH BEAM AS WELL AS AN END CONNECTION AT THE WALL.

NOTE: DIAPH. CALC. BECOMES MORE COMPLEX AT LOWER FLOORS BECAUSE THEY DISTRIBUTE FLOOR FORCES PLUS LOADS FROM SHEAR WALLS ABOVE ACCORDING TO RELATIVE RIGIDITIES OF SHEAR WALLS BELOW.

APPENDIX B
DIAPHRAGMS

B-1. Purpose and scope. The details and examples given in this appendix are to illustrate principles, factors, and concepts involved in seismic design of horizontal diaphragms of buildings. These are not mandatory, and other equivalent methods, materials, or details complying with this manual and applicable agency guide specifications may be used.

B-2. Design examples.

Design Example	Description
B-1	*Concrete Diaphragm:* floor diaphragm supported by steel deck; diaphragm stresses and stress transfer to concrete walls. See appendix A, design example A-1.
B-2	*Steel Deck Diaphragm:* stresses and connections in roof deck with concrete walls. See appendix A, design example A-1.
B-3	*Steel Deck Properties:* sample calculations for working shear, q_D, and flexibility factor, F, for six steel deck systems. See chapter 5, paragraph 5-6.
B-4	*Wood Diaphragm:* stresses and connections for diaphragms in a two-story wood building. See appendix A, design example A-6.

APPENDIX C
SHEAR WALLS

C-1. Purpose and scope. The data, details, commentary, and examples given in this appendix are to illustrate principles, factors, and concepts involved in seismic design of shear walls of buildings. These are not mandatory, and other equivalent methods, materials, or details complying with this manual and applicable agency guide specifications may be used.

C-2. Design examples.

Design Example	Description
C-1	*Concrete Shear Walls.* A detailed analysis and design of concrete shear walls is included in the design of a two-story building in appendix A, design example A-1, Box System.
C-2	*Concrete Shear Walls with Concrete Frame.* A special analysis for walls in buildings with K = 0.80 is included in the design of a two-story building in appendix A, design example A-4.
C-3	*Wood Stud Shear Walls.* An analysis and design of plywood and diagonally sheathed shear walls is included in the design of a two-story building in appendix A, design example A-6.
C-4	*Wall Stiffnesses.* Several methods of calculating wall rigidities are compared.

DESIGN EXAMPLE: C-4

COMPUTATION OF WALL STIFFNESSES:

The examples on sheet 3 through sheet 10 illustrate various methods for determining the rigidities of walls with openings parallel to plane of the wall.

(1) Method A and the first example is taken from a textbook, "Statically Indeterminate Structures," by J. R. Benjamin (Copyright 1959 by McGraw-Hill Book Company, Inc.), pages 221-223. It is a nearly precise method as it includes the effect of rotation of piers and axial shortening of piers. However, it does not include the effects of spandrel and foundation flexibilities. Computations made by this method are relatively accurate but can be very cumbersome for ordinary use.

(2) Method B_1 is a very commonly used method in which the total deflection of the wall is determined by adding the deflections of the piers at various levels. The piers are assumed to be fixed-ended or guided cantilevers depending on available restraints at pier ends. Joint rotation and axial shortening of piers is not considered.

(3) Method B_2 is the same as Method B_1, except that all piers are assumed to be fixed-ended.

(4) Method C is considered more accurate than either Method B_1 or B_2. In this method the deflection of wall is obtained as though it were a solid wall. From this is subtracted the deflection of that portion of the solid wall having the height of the openings. Then the deflection of actual piers at the openings is added, thus replacing the deflection of the fictitious solid midstrip. In this method the piers are assumed to be fixed-ended or cantilevers depending on available end restraints.

(5) Method D is a modification of Method B_2. Where a shear wall with openings is to be compared with a solid shear wall, the wall with openings is computed as in Method B_2 but the solid wall rigidity is computed by dividing the wall into horizontal strips each of the same vertical height as the strips used in the wall with openings. When comparing vertical resisting elements of various types this method may become confusing. However, where relative stiffnesses only are desired this is an improvement on Method B_2.

(6) A resume is given for the first example on sheet 7. This shows that Methods A, B_2, and C give comparable relative rigidities. If in the example, Piers B, C, and D each were of different proportions, there would be a slight difference in stiffnesses computed by Methods A and C. Method B_2 can produce inconsistent results. This is shown by the second example on sheets 8 and 9 in which Methods A, B_2, C, and D are compared. This shows consistent results between Methods A and C but for Method B_2 the wall with opening is

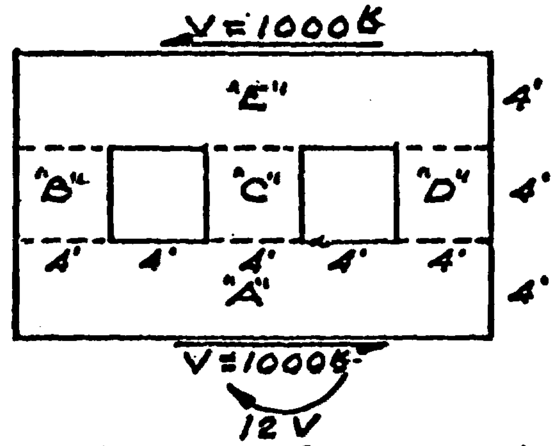
indicated as being stiffer than the solid wall. Method C is not generally as well-known as Methods B_1 and B_2 but is considerably more accurate and is used for the examples in Appendix A. The use of Method C to a more complex problem is shown on sheet 10.

COMPUTATION OF WALL STIFFNESSES

FIRST EXAMPLE: WALL WITH TWO OPENINGS

GIVEN:

THICKNESS $t = 8''$
MODULUS OF ELASTICITY
$E = 2{,}400\ ^{k}/_{in^2}$
MODULUS OF RIGIDITY
$G = 0.4E = 960\ ^{k}/_{in^2}$



WALL ELEVATION

METHOD A

ANALYSIS ACCOUNTING FOR ROTATION OF PIER "A" & CHANGE IN AXIAL PIER HEIGHT OF PIERS "E" "C.D"

REF. STATICALLY INDETERMINATE STRUCTURES, BY JACK R. BENJAMIN, 1959, pages 221-228

MOMENT & SHEAR DEFLECTION

PIER "A": $\Delta_A = \frac{Vh^3}{6EI}(2h+3e) + \frac{1.2Vh}{AG}$ $\quad I = \frac{t\,d^3}{12} = \frac{8(20)^3}{12} = 5340\ ^{IN\ FT^3}$

e IS DISTANCE FROM TOP OF WALL TO TOP OF PIER = 8'

$$\Delta_A = \frac{1000(4)^3(32)}{6 \times 2400 \times 5340} + \frac{1.2 \times 1000 \times 4 \times 12}{8 \times 20 \times 12 \times 960}$$

$$\Delta_A = 6.67 \times 10^{-3} + 31.2 \times 10^{-3} = 37.9 \times 10^{-3}\ \text{INCHES}$$

PIERS "B", "C", & "D": ASSUME $V = \frac{1000}{3} = 333^k$ & PIERS FIXED TOP & BOTTOM

$\Delta_{BCD} = \frac{Vh^3}{12EI} + \frac{1.2Vh}{AG}$ $\quad I = \frac{t\,d^3}{12} = \frac{8(4)^3}{12} = 42.7\ ^{IN\ FT^3}$

$$\Delta_{BCD} = \frac{333(4)^3}{12 \times 42.7 \times 2400} + \frac{1.2 \times 333 \times 4}{8 \times 4 \times 960}$$

$$\Delta_{BCD} = 17.3 \times 10^{-3} + 51.9 \times 10^{-3} = 69.2 \times 10^{-3}\ \text{INCHES}$$

PIER "E": $\Delta_E = \frac{Vh^3}{3EI} + \frac{1.2Vh}{AG}$

$$\Delta_E = \frac{1000(4)^3}{3 \times 2400 \times 5340} + \frac{1.2 \times 1000 \times 4}{8 \times 20 \times 960}$$

$$\Delta_E = 1.667 \times 10^{-3} + 31.2 \times 10^{-3} = 32.9 \times 10^{-3}\ \text{INCHES}$$

$$\Delta(\text{TOTAL}) = 0.140\ \text{INCHES}$$

ROTATION OF PIER "A"

$$\theta_A = \frac{Vh}{2EI}(h+2e) = \frac{1000\times4\times20}{2\times2400\times5140\times12} = .26\times10^{-3}\ \text{RADIANS}$$

Δ @ TOP OF WALL $6\times12\times0.26\times10^{-3} = 0.025''$

AXIAL DEFORMATION OF PIERS "B" & "D"

AXIAL FORCE AT MIDHEIGHT OF PIERS $F = \frac{6V}{16} = 375^K$

$$\Delta_h = \frac{Fh}{AE} = \frac{375\times4\times12}{8\times4\times12\times2400} = 0.0195''$$

$$\theta = \frac{2\Delta h}{16\times12}$$

$$\Delta \text{ @ TOP OF WALL} = \theta\times4\times12 = \frac{2\Delta h\times4\times12}{16\times12} = \frac{2\times.0195\times4\times12}{16\times12} = 0.00975''$$

TOTAL Δ @ TOP OF WALL $= 0.140 + 0.025 + 0.010 = 0.175''$

$$K = \frac{1}{0.175} = 5.72$$

METHOD B, REFER TO FIG. 6-11.

ANALYSIS BY STACKING PIERS AND USING DESIGN CURVES AND A COMMONLY USED METHOD OF ASSUMING WALL AS A WHOLE AS A CANTILEVER PIER.

PIER "E": $\frac{h}{d} = \frac{4}{20} = 0.20$ (CANTILEVER) CURVE 6

$$\Delta = 0.0175\times\frac{3000}{2400}\times\frac{12}{8} = 0.0328''$$

PIERS "B", "C", & "D": $\frac{h}{d} = 1.00$ (FIXED TOP & BOTT.) CURVE 4

$$\Delta_B = 0.111\times\frac{3000}{2400}\times\frac{12}{8} = 0.208 \qquad K_B = \frac{1}{0.208} = 4.81$$

$K_C = 4.81$

$K_D = 4.81$ +

$K_{BCD} = 14.43$

$$\Delta_{B,C,D} = \frac{1}{14.43} = 0.0694''$$

PIER "A": $\frac{h}{d} = \frac{4}{20} = 0.20$ (CANTILEVER) CURVE 6

$$\Delta = 0.0175\times\frac{3000}{2400}\times\frac{12}{8} = 0.0328''$$

Δ(TOTAL) $= 0.0328 + 0.0694 + 0.0328 = 0.135$ $K = 7.41$

NOTE: THAT THE STIFFNESS COMPUTED BY THIS METHOD INDICATES THAT THE WALL WITH WINDOW IS MORE RIGID THAN A SOLID WALL. (K = 7.22 - SEE METHOD C)

METHOD B_2 REFER TO FIG. G-11.

ANALYSIS BY STACKING PIERS AND USING DESIGN CURVES AND A COMMONLY USED METHOD OF ASSUMING ALL PIERS FIXED TOP & BOTTOM.

PIER E $\frac{h}{d} = 0.20$ (FIXED TOP & BOTT.) CURVE 4

$$\Delta = 0.01688 \times \frac{3000}{2400} \times \frac{12}{8} = 0.0316''$$

PIERS B, C, & D - SAME AS BEFORE - CURVE 4

$K_{BCD} = 14.43 \quad \Delta_{BCD} = 0.0694''$

PIER A $\frac{h}{d} = 0.20$ (FIXED TOP & BOTT) CURVE 4

$\Delta = 0.0316$

$\Delta(\text{TOTAL}) = 2 \times .0316 + .0694 = 0.1326 \qquad K = 7.54$

SOLID WALL $\frac{h}{d} = 0.60$ (FIXED TOP & BOTT.) CURVE 4

$$\Delta = 0.0560 \times \frac{3000}{2400} \times \frac{12}{8} = 0.105'' \qquad K = 9.52$$

$\boxed{\text{METHOD C}}$ REFER TO FIG. G-11.

ANALYSIS BASED ON DESIGN CURVE AND SUBTRACTION & ADDITION OF PIER INCREMENTS FROM SOLID WALL

Δ SOLID WALL $\frac{h}{d} = \frac{12}{20} = 0.60$

(CANTILEVER) CURVE 6

$\Delta = 0.074 \times \frac{3000}{2400} \times \frac{12}{8} = 0.1385''$

$K = \frac{1}{0.1385} = 7.22$ (STIFFNESS OF SOLID WALL)

Δ OF 4' HIGH MID STRIP OF WALL 4' × 20'

$\frac{h}{d} = \frac{4}{20} = 0.20$ CURVE 6

$\Delta = 0.0175 \times \frac{3000}{2400} \times \frac{12}{8} = 0.0328''$

INDIVIDUAL PIER B, C, OR D (FIXED TOP & BOTT.) CURVE 4

$\frac{h}{d} = 1.00$

$\Delta_B = 0.111 \frac{3000}{2400} \times \frac{12}{8} = 0.208''$ $K_B = \frac{1}{0.208} = 4.81$

$K_C = 4.81$

$K_D = 4.81+$

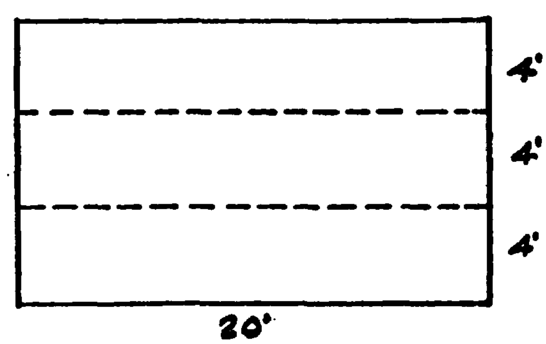
$\Delta_{B,C,D} = \frac{1}{14.43} = 0.0694''$ $K_{BCD} = 14.43$

$\Delta(\text{TOTAL}) = 0.1385 - 0.0328 + 0.0694 = 0.1751''$ $K = 5.72$

NOTE: THIS METHOD OF ANALYSIS IS ACCOMPLISHED IN THE DESIGN EXAMPLE C-1

$\boxed{\text{METHOD D}}$

ANALYSIS BASED ON COMPUTING THE STIFFNESS OF THE SOLID WALL BY BREAKING IT INTO THREE STRIPS, EACH 4' HIGH. THE STIFFNESS OF THE WALL WITH HOLES BEING ANALYZED PER METHOD B, ABOVE.



INDIVIDUAL STRIP

$\frac{h}{d} = 0.20$ (FIXED) CURVE 4

$\Delta = 0.0316''$

$\Delta(\text{TOTAL}) = 3 \times 0.0316 = \underline{\underline{0.0948''}}$

$K(\text{TOTAL}) = \frac{1}{0.0948} = \underline{\underline{10.54}}$

NOTE: THUS THE PROBLEM OF HAVING A WALL WITH HOLES BEING MORE RIGID THAN A WALL WITHOUT HOLES IS ELIMINATED.

PERCENTAGE OF RELATIVE STIFFNESS OF WALL WITH OPENINGS IN COMPARISON TO A SOLID WALL FOR THE METHODS "A" TO "D" ABOVE.

METHOD "A" $\frac{5.72 \times 100}{7.22} = 79.3\%$

METHOD "B_1" $\frac{7.41 \times 100}{7.22} = 102.6\%$

METHOD "B_2" $\frac{7.54 \times 100}{9.52} = 79.2\%$

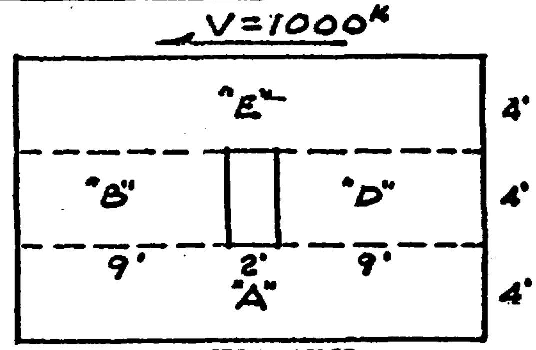
METHOD "C" $\frac{5.72 \times 100}{7.22} = 79.3\%$

METHOD "D" $\frac{7.54 \times 100}{10.54} = 71.5\%$

THUS METHOD "D" WOULD NOT GIVE QUITE AS MUCH SHEAR TO THE WALL WITH HOLES IN COMPARISION TO THE MORE THEORETICAL METHOD "A". IN ADDITION ONE COULD GET VERY INCONSISTENT RESULTS IF THERE WERE OTHER WALLS IN THE SYSTEM WITH MORE COMPLICATED ARRANGEMENTS OF OPENINGS. METHOD "C" CHECKS WELL WITH METHOD "A" BUT IS NO MORE TIME CONSUMING THAN METHODS "B" OR "D". IF, IN THIS EXAMPLE, PIERS B, C, AND D WERE EACH OF DIFFERENT PROPORTIONS, THERE WOULD BE SOME DIFFERENCE IN COMPARISON OF METHOD "A" TO "C". METHOD B_2 GIVES GOOD RESULTS FOR THIS EXAMPLE BUT FOR SOME PIER ARRANGEMENTS, INCONSISTENT RESULTS MAY BE OBTAINED.

COMPUTATION OF WALL STIFFNESSES - CONT.

SECOND EXAMPLE: WALL WITH ONE OPENING



GIVEN:

THICKNESS $t = 8''$

MODULUS OF ELASTICITY

$E = 2400^{K/IN^2}$

MODULUS OF RIGIDITY

$G = 0.4E = 960^{K/IN^2}$

$\boxed{\text{METHOD A}}$

PIER "A" CANTILEVER

$\Delta_A = 37.9 \times 10^{-3}$ INCHES

PIERS "B" & "D" $V = 500^K$

(FIXED TOP & BOTT.)

$V = 1000^K$

12V

$$\Delta_{BD} = \frac{500(4)^3}{12 \times 2400 \times 486} + \frac{1.2 \times 500 \times 4}{8 \times 9 \times 960}$$

$= 2.286 \times 10^{-3} + 34.72 \times 10^{-3} = 37.01 \times 10^{-3}$ IN. $\quad I = \frac{8(9)^3}{12} = 486^{IN\,FT^3}$

PIER "E"

$\Delta_E = 32.9 \times 10^{-3}$ IN.

$\Delta(\text{TOTAL}) = 0.1078''$

ROTATION OF PIER "A"

$\theta_A = .26 \times 10^{-3}$ RADIANS $\quad \Delta$ TOP OF WALL $= 0.025''$

AXIAL DEFORMATION - PIERS "B" & "D"

$F = \frac{6000}{11} = 545^K \quad \Delta_h = \frac{545 \times 4 \times 12}{8 \times 9 \times 12 \times 2400} = 0.01262''$

Δ TOP OF WALL $= \frac{2 \times .01262 \times 4}{11} = 0.0092''$

TOTAL Δ @ TOP OF WALL $= 0.1078 + .0250 + .0092 = .1420''$

WALL $K = \frac{1}{.1420} = 7.04$

STIFFNESS IN COMPARISON TO SOLID WALL $\frac{7.04 \times 100}{7.22}$

$= 97.5\%$

$\boxed{\text{METHOD } B_2}$

$\Delta_E = .0316, \ \Delta_B = \Delta_D = .0739 \quad K_B = 13.52 \quad K_{BD} = 27.04$

$\Delta_{BD} = .0370 \quad \Delta_A = .0316$

$\Delta_{WALL} = 2 \times .0316 + .0370 = .1002 \quad K_{WALL} = 9.98$

K SOLID WALL (FIXED) $= 9.52$

STIFFNESS IN COMPARISON

TO SOLID WALL

$\frac{9.98 \times 100}{9.52} = 104.8\%$

METHOD C

SOLID WALL (CANTILEVER) $\Delta = 0.1385"$
MIDSTRIP 4'x20' (CANT.) $\Delta = 0.0328"$

PIER B & D 4'x9' (FIXED)
$\Delta_B = \Delta_D = 0.0739"$ $K_B = K_D = 13.52$ $K_{BD} = 27.04$ $\Delta_{BD} = .0370"$

Δ WALL = .1385 − .0328 + .0370 = .1427 K WALL = 7.02

STIFFNESS IN COMPARISON TO SOLID WALL = $\frac{7.02 \times 100}{7.22} = 97.3\%$

METHOD D

$K_{WALL} = 9.98$ EQUIVALENT SOLID WALL $K = 10.54$
(SEG B_2) (SEE "D" OF FIRST EXAMPLE page 6 of 12)

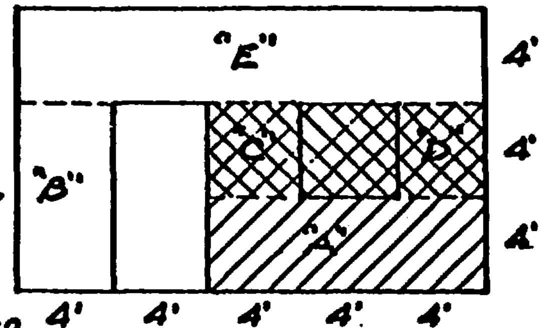
STIFFNESS IN COMPARISON TO SOLID WALL = $\frac{9.98 \times 100}{10.54} = 94.5\%$

AS IN THE FIRST EXAMPLE, METHODS "A" AND "C" CHECK WELL. METHOD "D" CHECKS METHOD "A" BETTER THAN IN THE FIRST EXAMPLE BUT THE COMPARATIVE STIFFNESS IS STILL SLIGHTLY LOW. HOWEVER, NOTE THAT IN THIS EXAMPLE METHOD "B", GIVES POOR RESULTS IN THAT THE WALL WITH A HOLE APPEARS STIFFER THAN THE SOLID WALL.

COMPUTATION OF WALL STIFFNESSES - CONT.

THIRD EXAMPLE: COMPLEX WALL BY METHOD C

REFER TO FIGURE G-11



GIVEN:
THICKNESS $t = 8''$
MODULUS OF ELASTICITY
$E = 2400\ ^{k}/in^{2}$

FROM SOLID WALL

Δ SOLID WALL: $\frac{h}{d} = \frac{12}{20} = 0.60$ (CANTILEVER) CURVE 6

$\Delta = 0.0740 \times \frac{3000}{2400} \times \frac{12}{8} = \underline{0.1385''}$ $K = \frac{1}{0.1385} = 7.22$
(STIFFNESS OF SOLID WALL)

Δ SOLID PIER ACD $\frac{h}{d} = \frac{8}{12} = 0.667$ (FIXED) CURVE 4

$\Delta = 0.0637 \times \frac{3000}{2400} \times \frac{12}{8} = 0.1195''$

Δ SOLID PIER CD $\frac{h}{d} = \frac{4}{12} = 0.333$ (FIXED) CURVE 4

$\Delta = 0.0287 \times \frac{3000}{2400} \times \frac{12}{8} = 0.0538''$

PIER "C" OR "D" $\frac{h}{d} = \frac{4}{4} = 1.0$ (FIXED) CURVE 4

$\Delta = 0.111 \times \frac{3000}{2400} \times \frac{12}{8} = 0.208''$ $K = \frac{1}{0.208} = 4.81$
4.81

Δ "C+D" $= \frac{1}{9.62} = 0.104''$ K C+D $= 9.62$

Δ "ACD" $= 0.1195 - 0.0538 + 0.104 = 0.1697''$

K ACD $= \frac{1}{0.1697} = 5.89$

Δ PIER "B" $\frac{h}{d} = \frac{8}{4} = 2.0$ (FIXED) CURVE 4

$\Delta = 0.389 \times \frac{3000}{2400} \times \frac{12}{8} = 0.73''$ $K = \frac{1}{0.73} = 1.37$

$K(B) + K(ACD) = 1.37 + 5.89 = 7.26$ $\Delta = \frac{1}{7.26} = \underline{0.1378}$

Δ' WALL $= \frac{h}{d} = \frac{8}{20} = 0.4$ (CANTILEVER) CURVE 6

$\Delta' = 0.0405 \times \frac{3000}{2000} \times \frac{12}{8} = \underline{0.0758''}$

Δ WALL (TOTAL)
$= 0.1385 - 0.0758 + 0.1378$
$= 0.2005''$

$K_{WALL} = \frac{1}{0.20005} = 4.99$

Example C-4 | 10 of 10 | Wall Stiffnesses

APPENDIX D
SPACE FRAMES

D-1. Purpose and scope. The data, details, and examples given in this appendix are to illustrate principles, factors, and concepts involved in seismic design of moment resisting space frames of buildings. These are not mandatory, and other equivalent methods, materials, or details complying with this manual and applicable agency guide specifications may be used.

D-2. Design examples.

Design Example	Description
D-1	*Concrete Ductile Moment Resisting Space Frames.* Illustrates special analyses required to design ductile moment resisting frames using reinforced concrete. See appendix A, design example A-2.
D-2	*Steel Ductile Moment Resisting Space Frames.* Illustrates special analyses required to design ductile moment resisting frames using structural steel. See appendix A, design example A-3.

APPENDIX E
REINFORCED MASONRY

E-1. Purpose and scope. The data, details, and examples given in this appendix are to illustrate principles, factors, and concepts involved in seismic design of reinforced masonry buildings. These are not mandatory, and other equivalent methods, materials, or details complying with this manual and applicable agency guide specifications may be used.

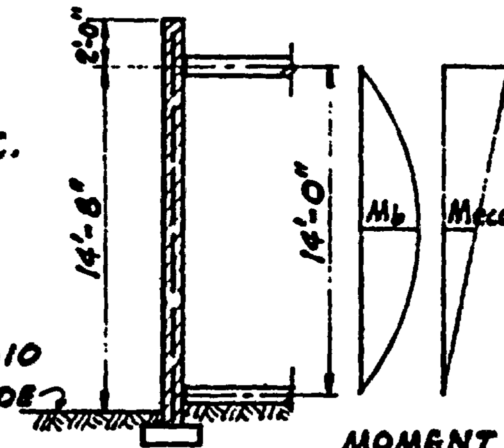
E-2. General. Design methods are similar to those for concrete. For details of masonry construction, see chapter 8.

E-3. Design examples.

Design Example	Description
E-1	*Wall Design—Lateral Load Normal to Wall*
E-2	*Wall Design—Lateral Load Parallel to Wall (Shear Wall)*
E-3	*Composite Wall*
E-4	*Wall Stiffnesses.* For calculation of wall stiffnesses see appendix C, design example C-4.
E-5	*Shear Wall Buildings.* For design of a shear wall building see appendix C, design example C-1.

WALL DESIGN

LATERAL LOAD NORMAL TO WALL



GIVEN: 8" LT. WT. CMU W/#5 @ 24" O.C.
ROOF DL = 300#/FT
LL = 100#/FT
ROOF ECCENTRICITY = 5"
f'_m = 1350 PSI TABLE 8-2
WALL WT. = 53#/SF TABLE 8-10
ZONE 4

WIND = 15#/SF MIN.

SEISMIC: $F_p = Z I C_p W$

$= 1.0 \times 1.0 \times 0.3 \times 53\#/SF = 15.9\#/SF$ GOVERNS

BENDING MOMENT $M = \frac{wL^2}{8} = \frac{15.9 \times 14.67^2}{8} \times 12 = 5136''\#$ (NEGLECT MOMENT DUE TO PARAPET CANT.)

ECCENTRIC MOMENT @ MID-HEIGHT $M_{ecc} = 300\# \times \frac{5''}{12} \times \frac{1}{2} = 62.5''\#$

TOTAL $M = 5136 + 62.5 = 5199''\# \times 2'$ VERT ELEMENT $= 10400''\#$

$A_s = 0.31$ $\quad p = \frac{A_s}{bd} = \frac{0.31}{24 \times 3.8} = .0034$; $n = \frac{E_s}{E_m} = \frac{29 \times 10^6}{1.35 \times 10^6} = 22$; $np = 0.075$

$$k = \sqrt{2np + (np)^2} - np = \sqrt{2(.075) + (.075)^2} - .075 = 0.319$$

$$j = 1 - \frac{k}{3} = 0.8936$$

$$f_m = \frac{2M}{bd^2jk} = \frac{2 \times 10400}{24 \times (3.8)^2 \times 0.8936 \times 0.319} = 210 \text{ PSI}$$

$$f_s = \frac{M}{A_s jd} = \frac{10400}{0.31 \times 0.8936 \times 3.8} = 9880 \text{ PSI} < 20{,}000 \text{ PSI}$$

OK

Example E-1 | 1 of 2 | Normal to Wall

AXIAL LOAD @ MID-HEIGHT ROOF DL = $300^{\#} + 53^{\#} \times \left(\frac{14}{2} + 2'\right) = 777^{\#}/_{1}$

AREA EFFECTIVE IN COMPRESSION = $(7.5'' \times 7.625) + (1.25 \times 2 \times 16.5) = 98.4^{\square''}$

$$f_a = \frac{P}{A} = \frac{777 \times 2'}{98.4} = 15.8 \text{ PSI}$$

$$F_a = 0.2 f_m \left[1 - \left(\frac{h}{40t}\right)^3\right] = 0.2(1350)\left[1 - \left(\frac{14.67 \times 12}{40 \times 7.62}\right)^3\right] = 218 \text{ PSI}$$

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} = \frac{15.8}{218} + \frac{210}{450} = 0.539 < 1.33 \quad \text{OK}$$

CONNECTION @ ROOF DIAPHRAGM:

REACTION = $15.9^{\#}(7+2) = 143^{\#}/_{1}$

MIN. REACTION $200^{\#}/_{1}$

PARAPET DESIGN: $F_p = ZISC_pW$

$= 1.0 \times 1.0 \times 1.5 \times 0.8 \times 53^{\#} = 63.6^{\#}/SF$

$$M = 63.6 \times \frac{2^2}{2} \times 12 = 1526''^{\#}$$

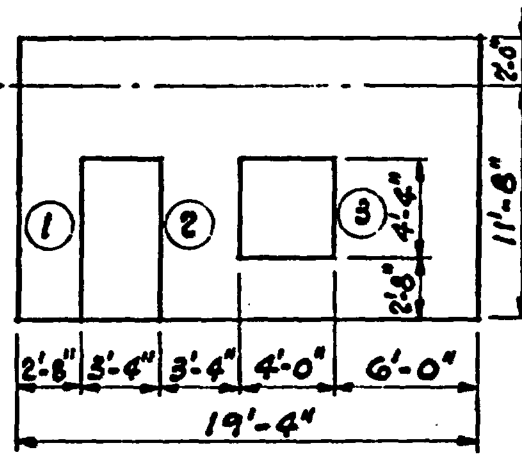
$$f_m = \frac{2M}{bd^2jk} = \frac{2 \times 1526}{24 \times (3.8)^2 \times 0.8936 \times 0.319} =$$

$31 \text{ PSI} < 450 \times 1\frac{1}{3}$ OK

SHEAR WALL

LATERAL LOAD PARALLEL TO WALL

GIVEN: LATERAL LD = 10^K $F = 10^K \rightarrow$
8" LT.WT. CMU
#5 @ 24" O.C.



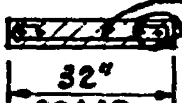
THE CALCULATIONS FOR PIER RIGIDITY ARE SIMILAR TO EXAMPLE C-4.

PIER 1

FLEXURAL LOAD

% LATERAL FORCE TO PIER 1 = 9% × 10^K = 0.9^K

TIES PER TABLE 8-7
2-#5



32"
SOLID GROUT

INCREASE 50% FOR SEISMIC

$A_s = 0.62$

$v = \frac{V}{bd} = \frac{900 \times 1.5}{7.62 \times 24} = 7.4\ PSI < 35 \times 1.33$

$p = \frac{0.62}{7.62 \times 24} = .0034$

$np = 22 \times .0034 = .075$

$k = 0.319 \quad j = 0.894$

$fb = \frac{2M}{bd^2jk} = \frac{2 \times 3150 \times 12}{7.62 \times 24^2 \times .894 \times .319} = 60$ PSI FLEXURAL STRESS

AXIAL LOAD

ROOF D.L. = $300^{\#}/_{l}$ × 4.33' TRIB = $1299^{\#}$/PIER

WALL WT. @ MID HT PIER = $77^{\#}(8.5 \times 2.67 + 6.67' \times 4.33)$ = $2943^{\#}$/PIER

$P = 1299 + 2943 = 4242^{\#}$/PIER

$f_a = \frac{P}{A} = \frac{4242}{32 \times 7.62} = 17$ PSI AXIAL STRESS

$F_a = 0.2 f'm \left[1 - \left(\frac{h}{40t}\right)^3\right] = 0.2(1350)\left[1 - \left(\frac{11.67 \times 12}{40 \times 7.62}\right)^3\right] =$

244 PSI ALLOWABLE AXIAL STRESS

AXIAL STRESS DUE TO OVERTURNING OF WALL PANEL

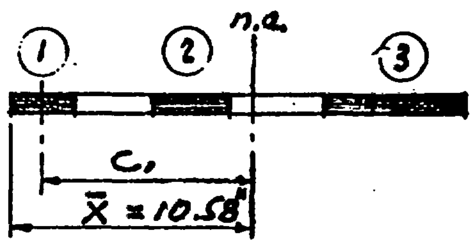
$M_o = 10{,}000^{\#} \times 11.67 = 116{,}700'^{\#}$

$$f_o = \frac{Mc}{I} = \frac{116700 \times 9.25}{454 \times \frac{7.62}{12} \times 144} = 26 \text{ PSI}$$

FLEXURAL + AXIAL + OVERTURNING

$$\frac{60}{450} + \frac{17}{244} + \frac{26}{244} = 0.31 < 1.33$$

OK



FIND n.a. OF WALL

PIER	AREA	X	AX	C	Ac^2
1	2.67t	1.33	3.55t	9.25	228t
2	3.33t	7.67	25.5t	2.91	28t
3	6.0 t	16.33	98.0t	5.75	198t
	Σ 12t		Σ127t		Σ454t = I

$$\bar{X} = \frac{127}{12} = 10.58''$$

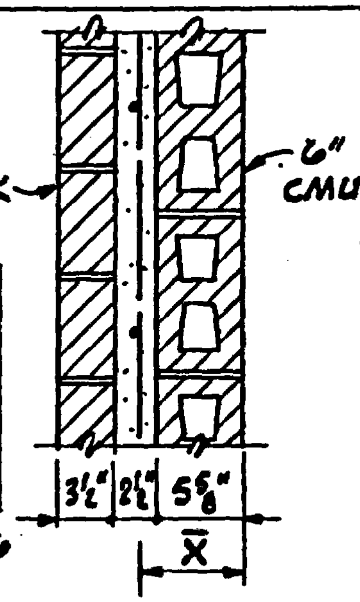
PIER	% OF LAT. LD.	FORCE TO EA. PIER	bd	v PSI	$M = \frac{Fh}{2}$	f_b	f_a	f_o
1	9	900#	244□" SOLID GROUT	3.7	3150'#	60	17	26
2	26	2600	305□" SOLID GROUT	8.5	5629	67	20.5	9.
3	65	6500	450□" SOLID GROUT	11.8	14070	54	13.7	1.6
	100	10,000#						

Example E-2 2 of 2 Parallel to Wall

COMPOSITE WALL

BRICK/CMU

TO COMPUTE THE CENTER OF RESISTANCE FOR AXIAL LOAD



	AREA □"	$E \times 10^6$	$EA \times 10^6$	$\bar{X}$	EAX
BRICK	$3\frac{1}{2} \times 16 = 56$	$\times 1.5$	84	9.87	829
GROUT	$2\frac{1}{2} \times 16 = 40$	$\times 2.0$	80	6.87	549.6
CMU	$50^{*} = 50$	$\times 1.35$	67.5	2.8	189
			$\Sigma = 231.5$		$\Sigma = 1567.6$

$$\bar{X} = \frac{\Sigma EAX}{\Sigma EA} = \frac{1567.6}{231.5} = \underline{6.77}$$

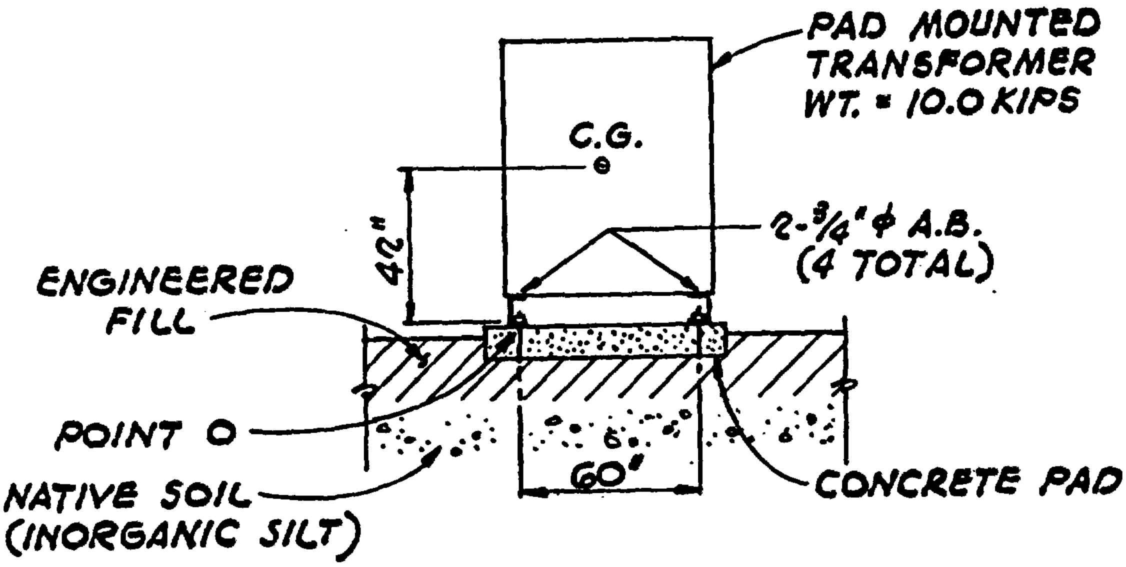
* FROM TABLE 8-9

APPENDIX F
MECHANICAL AND ELECTRICAL ELEMENTS

F-1. Purpose and scope. The design examples in this appendix are to illustrate principles, factors and concepts involved in seismic design. These are not mandatory; and other equivalent methods, materials or details complying with this manual and applicable agency guide specifications may be used.

F-2. Design examples:

Design Example	Description
F-1	*Pad-Mounted Transformer:* Illustrates the seismic design of a typical, rigidly mounted item of equipment on the ground.
F-2	*Pole-Mounted Transformer:* Illustrates the application of the provisions of paragraph 10-5 to the seismic analysis of flexible equipment on the ground.
F-3	*Tower-Mounted Equipment:* Tower-supported equipment is investigated for lateral seismic loads. The tower period is computed.
F-4	*Cooling Tower in Building:* Presents analysis for a rigidly mounted cooling tower in a multi-story building.
F-5	*Unit Heater—Flexible Brace:* Analysis of a unit heater not rigidly braced.
F-6	*Unit Heater—Rigid Support:* Demonstrates the reduction of the lateral seismic load by rigidly bracing the unit heater of Design Example F-5.
F-7	*Water Heater:* Indicates how a water heater in a barracks is investigated for seismic loads.
F-8	*Tank on a Building:* Demonstrates the seismic analysis of a storage tank on a building. Emphasis is placed on the period determination.
F-9	*Water Riser:* Illustrates an approximate scheme used to determine the seismic loading on pipe connections. A riser in a multi-story building is treated.



GIVEN: W = 10.0 KIPS
RIGID EQUIPMENT ON THE GROUND
ZONE 3 SEISMIC AREA AND I = 1.0

REQUIRED: CHECK ANCHOR BOLT REACTIONS DUE TO SEISMIC LOADS.

SOLUTION:

$$F_p = ZI\,(2/3\,C_p)W_p \qquad (10\text{-}3)$$

$Z = 3/4,\ I = 1.0,\ C_p = 0.30,\ W_p = 10.0$ KIPS

$F_p = 3/4\,(1.0)(2/3)(0.30)(10) = 1.5$ KIPS
APPLIED AT CG

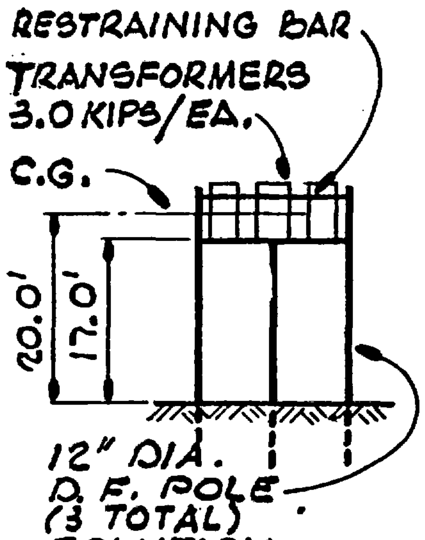
SHEAR/BOLT = 1.5/4 = 0.38 KIPS/BOLT
ALLOW. SHEAR = 1.50 KIPS/BOLT
∴ 4-3/4"φ A.B. O.K.

CHECK OVERTURNING -
$\Sigma M_O = 0$
$42'' \times 1.5^k << \frac{60''}{2} \times 10.0^k$ ∴ OVERTURNING O.K.

Reference: Chapter 10, paragraph 10-5a

Design Example F-1 | 1 of 1 | Pad Mounted Transformer

TM 5-809-10
NAVFAC P-355
AFM 88-3, Chap. 13



GIVEN:

WT. TRANSFORMERS = 3.0 KIPS / EA.
WT. POLES ——— 35 LB/FT./POLE
E (POLES) ——— 1.6×10^6 LB/IN.^{2}
T_S (SITE PERIOD) IS UNKNOWN
ASSUME EACH POLE ACTS AS A 20' LONG CANTILEVER
SEISMIC ZONE 3 & HIGH RISK

REQUIRED:

FIND THE SEISMIC FORCE COEFFICIENT FOR THE WEAK AXIS OF THE POLE FRAME. (I.E., NORMAL TO THE PAPER.)

SOLUTION:

EQUIPMENT ON GROUND, FLEXIBLY MOUNTED. CLASSIFY AS STRUCTURE OTHER THAN BUILDING. (PARA. 10-5c), INVERTED PENDULUM (PARA. 11-3).

$$T = 0.32\sqrt{\frac{W}{k}} \qquad (10\text{-}1)$$

$$W = 3000 + \frac{35 \times 20}{2} = 3{,}350 \text{ LB/POLE}$$

CALCULATION OF k:

$$I_o \text{ (ONE POLE)} = .785R^4 = .785(6)^4 = 1030 \text{ IN.}^4$$

$$\Delta = \frac{PL^3}{3EI_o} \quad \text{OR} \quad k = \frac{3EI_o}{L^3} = \frac{3(1.6 \times 10^6)(1030)}{(20 \times 12)^3}$$

$$= 354 \text{ LBS/IN.}$$

$$\therefore T = 0.32\sqrt{\frac{3350}{354}} = 0.98 \text{ SEC.}$$

$$F_p \equiv V = ZIKCSW \qquad (3\text{-}1)$$

$Z = \frac{3}{4}$ (ZONE 3), $I = 1.25$ (HIGH RISK),
$K = 2.5$ (INVERTED PENDULUM), $S = 1.5$
(T_S NOT KNOWN)

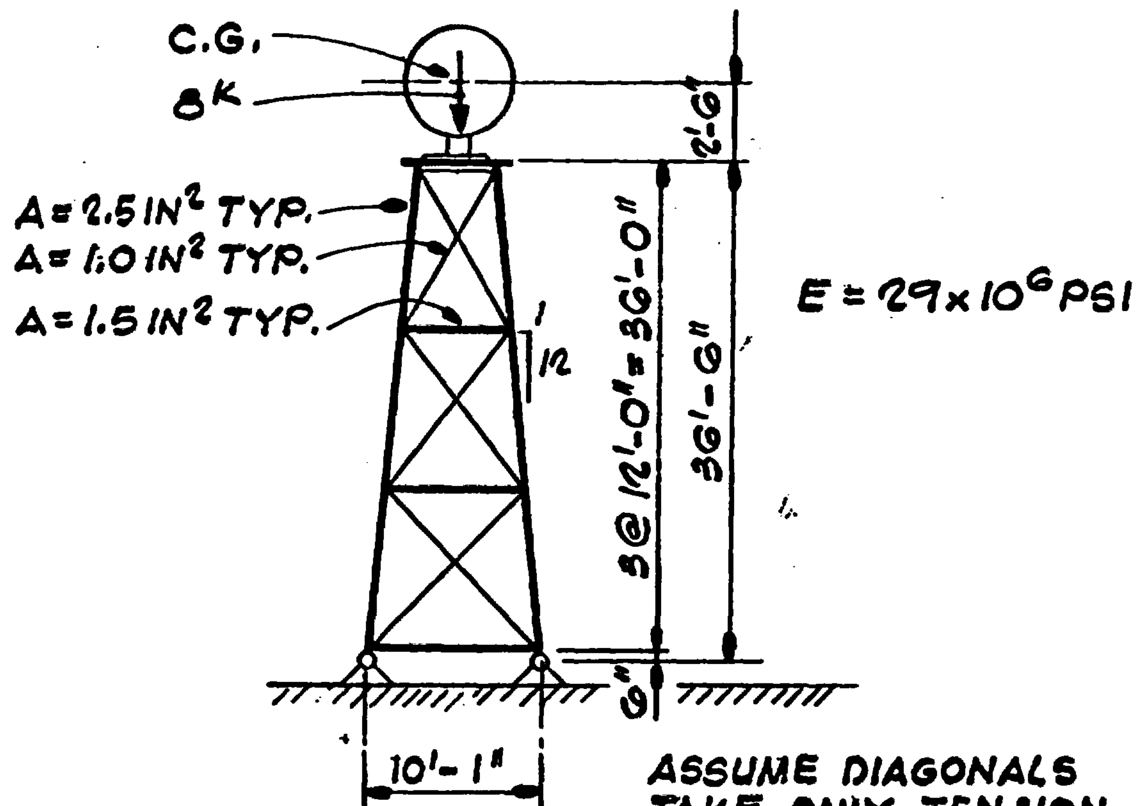
$$C = 1/15\sqrt{T} = 0.067 \text{ (FORMULA 3-2)}$$

$$F_p = \tfrac{3}{4} \times 1.25 \times 2.5 \times 0.67 \times 1.5W = \underline{\underline{0.236\,W}}$$

Design Example F-2 | 1 of 1 | Pole Mounted Transformer

GIVEN:

MISSILE TRACKING DEVICE SITUATED ON TRUSS TOWER: SEISMIC ZONE 2, ESSENTIAL FACILITY
T_s (SITE PERIOD) = 2.0 SEC.



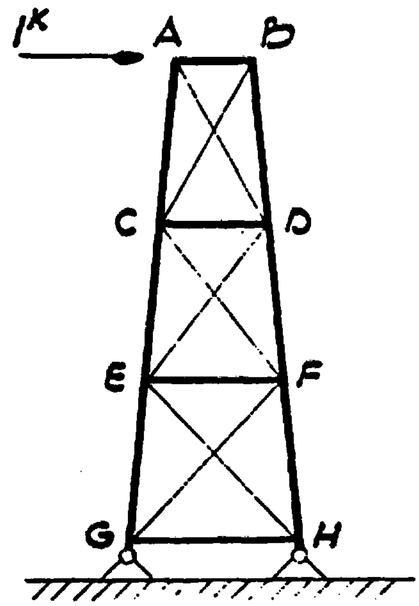
ASSUME DIAGONALS TAKE ONLY TENSION

REQUIRED:

FIND THE LATERAL SEISMIC FORCE TO BE APPLIED AT THE CENTER OF GRAVITY OF THE TRACKING DEVICE. CLASSIFY AS RIGID EQUIPMENT ON A STRUCTURE OTHER THAN A BUILDING (PARA. 10-5c), INVERTED PENDULUM (PARA. 11-3).

SOLUTION:

MEMBER	P FORCE (KIPS)	L (IN.)	A (IN.^{2})	$\frac{P^2L}{A}$
AB	1.00	48	∞	0
AC	0	145	2.5	0
AD	0	156	1.0	0
BC	+2.17	156	1.0	734.6
BD	-2.02	145	2.5	236.6
CD	-0.67	72	1.5	21.5
CE	+2.02	145	2.5	236.6
CF	0	167	1.0	0
DE	+1.16	167	1.0	224.7
DF	-3.02	145	2.5	529.0
EF	-0.50	96	1.5	16.0
EG	+3.02	145	2.5	529.0
EH	0	180	1.0	0
FG	+0.75	180	1.0	101.8
FH	-3.63	145	2.5	764.3
GH	+0.30	120	1.5	7.2



NOTE: PT. H IS ASSUMED TO TAKE NO BASE SHEAR AS MEMBER EH CARRIES NO LOAD.

$$1^K \cdot \frac{\Delta}{2} = \sum \frac{P^2L}{2AE} \; ; \; \sum \frac{P^2L}{A} = 3401.3 \; K^2/IN.$$

$$\sum \frac{P^2L}{AE} = 1.17 \times 10^{-1} = 0.117 \text{ INCHES/KIP}$$

$$\left(\frac{1}{\Delta}\right) = k \quad k = 8.55 \text{ KIPS/IN. PER SIDE}$$

$$T = 0.32\sqrt{\frac{W}{k}} = 0.32\sqrt{\frac{8.0}{2(8.55)}} = 0.22 \text{ SEC.} \; (10\text{-}1)$$

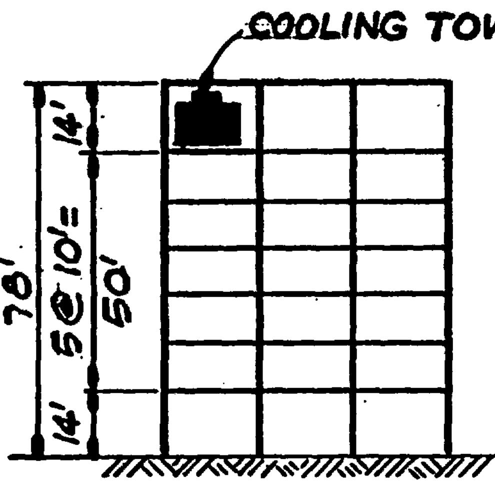
$Z = \frac{3}{8}$ (ZONE 2), $I = 1.5$ (ESSENTIAL FACILITY)

$K = 2.5$ (INVERTED PENDULUM), $CS = 0.137$ (TABLE 4-3)

$F_p = V = ZIKCSW = \frac{3}{8} \times 1.5 \times 2.5 \times 0.137 = 0.193\,W$

$= 0.193 \times 8 = \underline{\underline{1.54}}$ KIPS

NOTE: WEIGHT OF TOWER WAS NEGLECTED.



GIVEN:

WT. COOLING TOWER = 20.0 KIPS
ZONE 3 SEISMIC AREA
CONSIDER TOWER RIGIDLY MOUNTED
WT. TYP. FLOOR = 400 KIPS
100% MOMENT RESISTING FRAME. I = 1.0.

REQUIRED:

FIND THE SEISMIC DESIGN FORCE TO BE APPLIED AT C.G. OF COOLING TOWER.

SOLUTION:

CHECK MASS RATIOS (PARA. 10-2c)

W_p/w_x FLOOR $20/400 < 0.20$ O.K.

W_p/W STRUCT. $20/2800 < 0.10$ O.K.

QUALIFIES AS RIGID EQUIPMENT, RIGIDLY MOUNTED IN A BUILDING (PARA. 10-3).

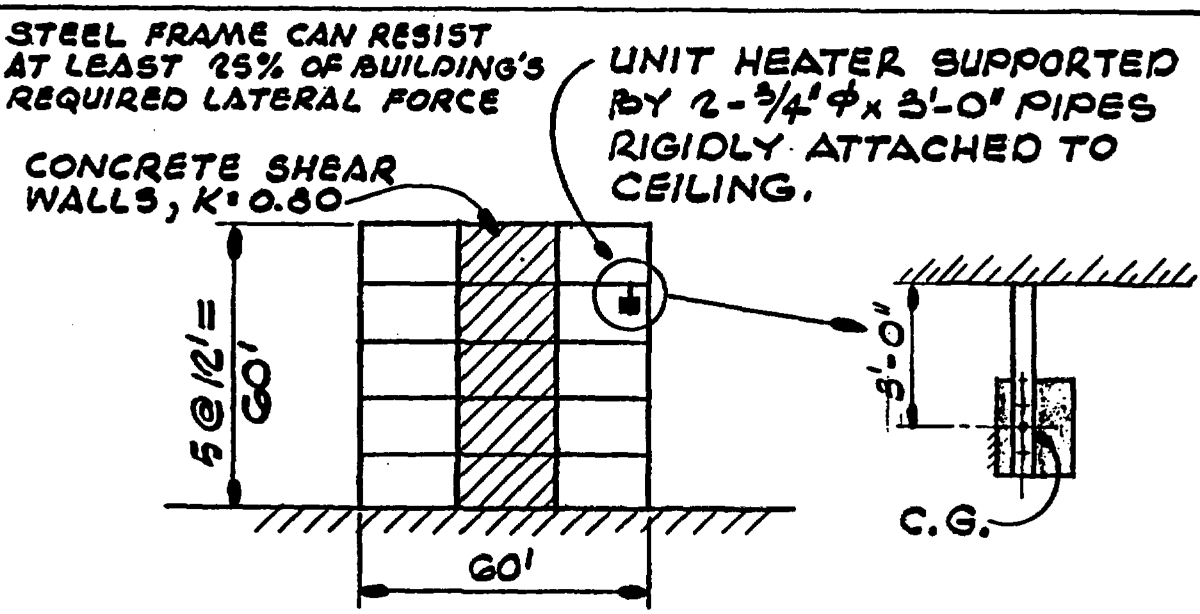
$$F_p = Z I C_p W_p \qquad (3-8)$$

$$Z = \tfrac{3}{4} \text{ (ZONE 3)}, \quad I = 1.0$$

$$C_p = 0.30 \text{ (TABLE 3-4)}$$

$$F_p = \tfrac{3}{4} \times 1.0 \times 0.30 \times W_p = 0.225\, W_p$$
$$= 0.225 \times 20 = \underline{\underline{4.5 \text{ KIPS}}}$$

Design Example F-4 | 1 of 1 | Cooling Tower in Buildings



GIVEN: NEGLECT EFFECTS OF ROTATION OF UNIT HEATER.

W_p = WT. UNIT HEATER = 350 LBS

w_x = WT. TYPICAL FLOOR = 500 KIPS

W = WT. STRUCTURE = 2300 KIPS

I (OCCUPANCY) = 1.0

ZONE 3 SEISMIC AREA

I_o (3/4"Φ PIPE) = 0.037 IN^4

E (PIPE) = 30×10^3 $KIPS/IN^2$

REQUIRED: FIND DESIGN SEISMIC FORCE TO BE APPLIED AT C.G. OF UNIT HEATER.

SOLUTION: CHECK MASS RATIOS: (PARA. 10-2c)

W_p/w_x FLOOR = $0.35/500 \ll 0.20$ OK

W_p/W STRUCT. = $0.35/2300 \ll 0.10$ OK

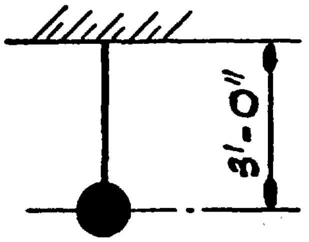
(EQ: 8-10-4)

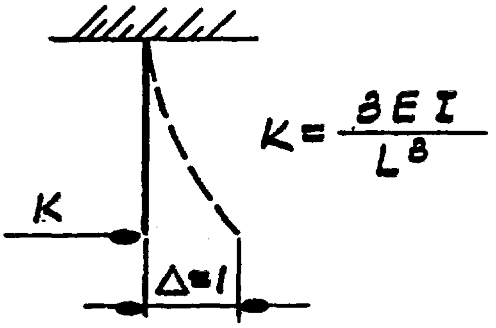
INVESTIGATE AS FLEXIBLY MOUNTED EQUIPMENT IN BUILDINGS PARA. 10-4

$$F_p = Z I A_p C_p W_p \qquad (10\text{-}2)$$

Design Example F-5 | 1 of 2 | Unit Heater - Flexible Brace

Z = 3/4 (ZONE 3), I = 1.0, C_p = 0.30 (TABLE 3-4)
A_p, WHICH RANGES FROM 1.0 TO 5.0 IS DEPENDENT ON PERIODS T_a (EQUIP.) AND T (BLDG.)
REFER TO PARA. 10-4e





$$k = 2\left\{\frac{3(30\times10^3)(0.037)}{36^3}\right\} = 0.142 \text{ KIPS/INCH.}$$

$$T_a = 0.32\sqrt{\frac{W_p}{k}} = 0.32\sqrt{\frac{.35}{.142}} = 0.50 \text{ SEC.} \qquad (10\text{-}1)$$

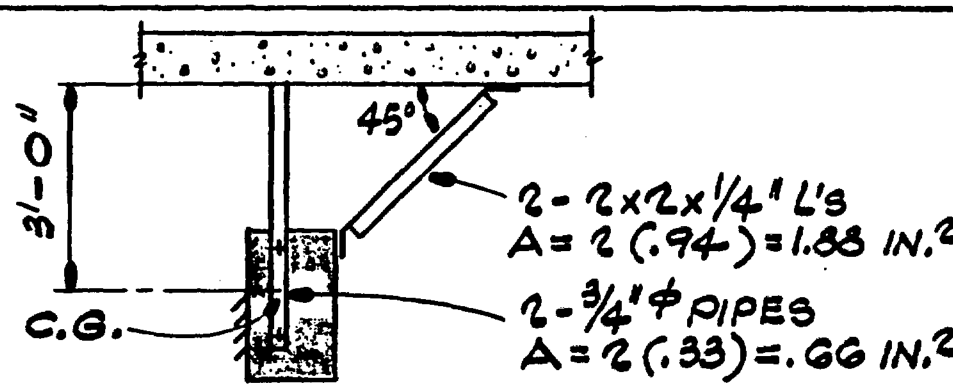
T = 0.6 SEC (FROM ANALYSIS OF BUILDING, REFER TO PARA. 10-4c(1))

$$\frac{T_a}{T} = \frac{0.50}{0.60} = 0.83$$

USE FIGURE 10-3b: A_p = 4.90 (TABLE 10-1)

$$F_p = 3/4 \times 1.0 \times 4.9 \times 0.30\, W_p = 1.10\, W_p$$
$$= 1.10 \times 350 = \underline{\underline{386 \text{ LBS.}}}$$

NOTE: A LATERAL FORCE OF 386 LBS. WILL OVERSTRESS THE 3/4"φ PIPE BRACES; THEREFORE ADD DIAGONAL SUPPORTS AS SHOWN IN DESIGN EXAMPLE F-6



DETAIL OF UNIT HEATER

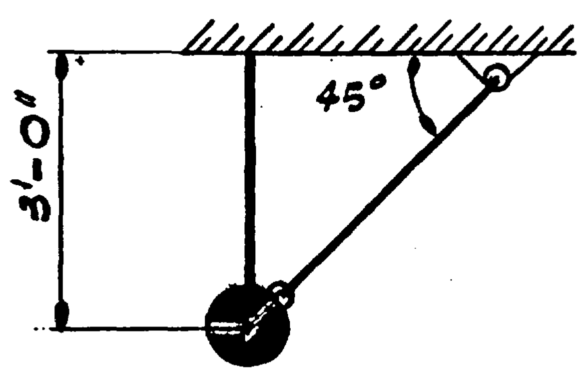
GIVEN : USE DATA GIVEN IN DESIGN EX. F-5

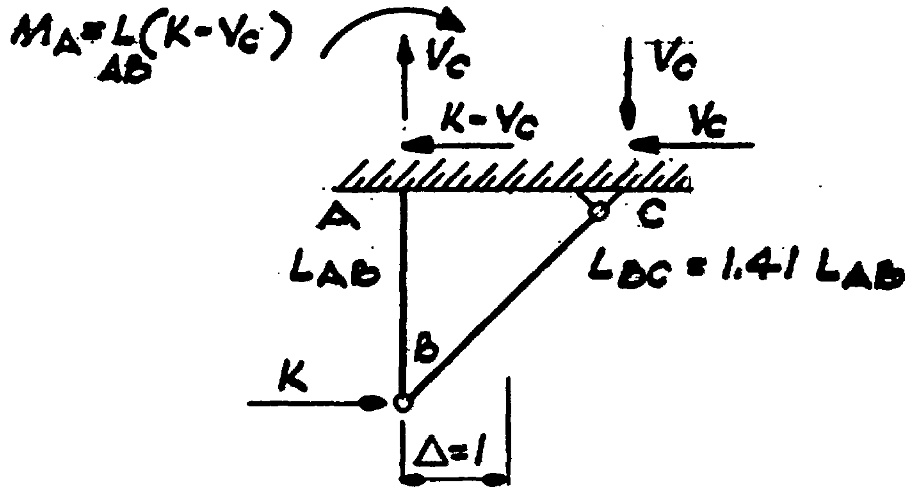
REQUIRED : FIND DESIGN SEISMIC FORCE

SOLUTION : $F = ZIC_pW_p$ (3-8)
IF RIGIDLY MOUNTED, PARA. 10-3

CALCULATION OF T_a FOR RIGIDITY CHECK:

APPROXIMATE ANGLE CONNECTIONS BY PINS. ASSUME ALL LATERAL FORCE IS RESISTED BY BRACING ANGLES. USE ENERGY METHOD TO CALC. K_2.





ASSUME $K - V_C \doteq 0$: THIS ASSUMES ALL OF THE HORIZONTAL FORCE K IS RESISTED BY THE DIAGONAL.

$$\Sigma W_{\text{EXTERNAL}} = \Sigma W_{\text{INTERNAL}}$$

$$K\left(\frac{\Delta}{2}\right) = \frac{K^2 L_{AB}}{2 A_{AB} E} + \frac{(1.41K)^2 L_{BC}}{2 A_{BC} E}$$

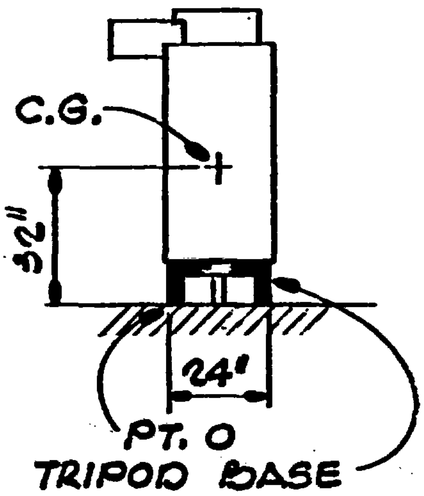
$$1 = K\left(\frac{L_{AB}}{A_{AB} E} + \frac{1.41^3 L_{AB}}{A_{bc} E}\right)$$

$$K = \frac{30 \times 10^6}{\left(\frac{36}{0.66} + \frac{1.41^3(36)}{1.88}\right)} = 2.78 \times 10^5 \text{ LBS/INCH}$$

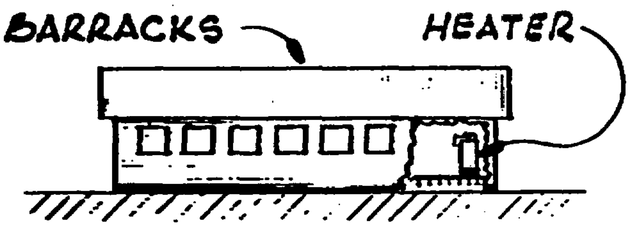
$$T_a = 0.32 \sqrt{\frac{350}{2.78 \times 10^5}} = 0.011 \text{ SEC.} \qquad (10\text{-}1)$$

$T_a < 0.05$ SEC., THEREFORE SUPPORT IS RIGID (PARA. 10-3)

$$F_p = Z I C_p W_p = 3/4 \times 1.0 \times 0.30\, W_p = 0.225\, W_p$$
$$= 0.225 \times 350 = \underline{\underline{79 \text{ LBS.}}}$$



GIVEN: 1445 LB. WATER HEATER IN BARRACKS, SEISMIC ZONE 4.



REQUIRED: INVESTIGATE THE WATER HEATER FOR SEISMIC LOADS.

SOLUTION: WATER HEATER WILL BE CLASSIFIED AS BEING EQUIPMENT ON THE GROUND AND WILL BE CONSIDERED TO BE A RIGID BODY. SINCE FRICTION CANNOT BE USED TO RESIST LATERAL SEISMIC FORCES, THE WATER HEATER MUST BE RIGIDLY ATTACHED TO ITS FOUNDATION. BOLT WATER HEATER LEGS TO FLOOR. REFER TO PARA. 10-5a

$$F_p = ZI\,(2/3\ C_p)\,W_p \qquad (10\text{-}3)$$

$Z = 1.0$, $I = 1.0$, $C_p = 0.30$ (TABLE 3-4)

$$F_p = 1.0 \times 1.0 \times 2/3 \times 0.30 = 0.20W_p$$

$F_p = 0.20\,W_p = 0.20 \times 1.445 = 0.29$ KIPS

$F_p = 0.29K$ APPLIED AT C.G.

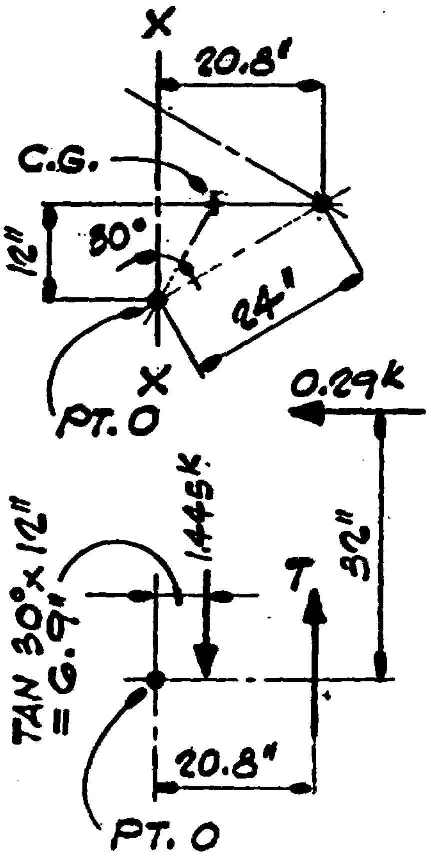
CHECK FOR OVERTURNING ABOUT POINT O.

$\Sigma M_{x-x} = O$

$0.29^K \times 32'' < 1.445^K \times \text{TAN.}\,30° \times 12''$

$9.28''^K < 10.0''^K$

OVERTURNING O.K.



CHECK FOR LOAD T IN LEG OF TRIPOD.

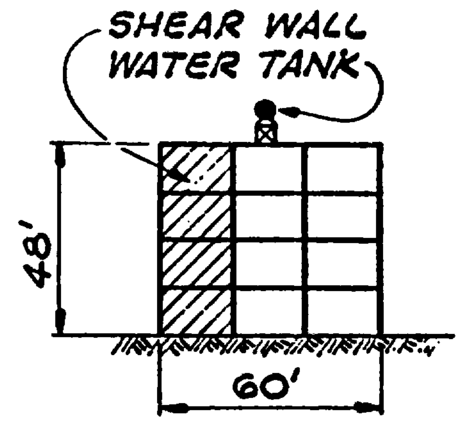
$\Sigma M_{x-x} = O = T \times 20.8 + 0.29 \times 32 - 1.445^K \times \text{TAN}\,30° \times 12''$

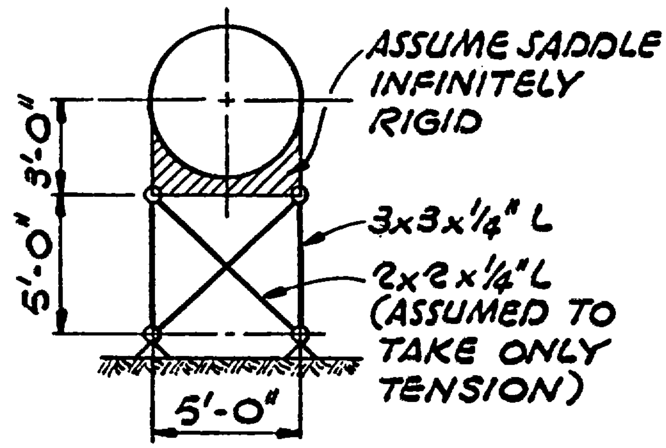
$T = \dfrac{-9.28''^K + 10.0''^K}{20.8''} = 0.035$ KIPS COMPRESSION

HENCE, USE NOMINAL ANCHOR BOLTS. USE 3-5/8" ϕ A.B.

ALLOW BASE SHEAR = $3(1.0^K) = 3K$

SHEAR O.K. 3.0K >> 0.29 K





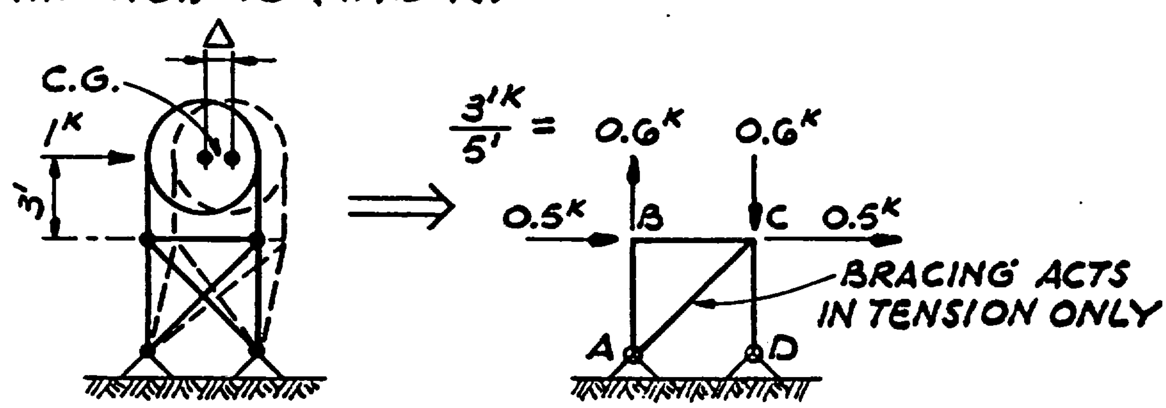
DETAIL OF TANK SUPPORT

GIVEN: WT. OF TANK + WATER = 10.0 K/ TRUSS

ZONE 2 SEISMIC AREA AND I = 1.0 OCCUPANCY
ASSUME ALL JOINTS ARE PIN CONNECTIONS.
ASSUME CROSS MEMBERS TAKE TENSION ONLY.
NEGLECT WT. OF SUPPORT MEMBERS.

REQUIRED: FIND THE DESIGN SEISMIC FORCE.

SOLUTION: HYDRO-DYNAMIC EFFECTS ARE NEGLECTED EVEN WHEN TANK IS PARTIALLY FULL. CALCULATION OF STIFFNESS OF TANK STRUCTURE: USE ENERGY METHOD TO FIND K.



COMPUTATION OF Δ : $1^K \cdot \frac{\Delta}{2} = \Sigma \frac{F^2 L}{2AE}$

MEMBER	LENGTH	AREA	F	F^2L/A
AB	5.00 FT.	1.44 IN.^{2}	+ .6 K	1.25
CD	5.00	1.44	− 1.6	8.89
CA	7.07	0.94	+ 1.414	15.03
				25.17

$$\therefore 1K \times \left(\frac{\Delta}{2}\right) = \frac{25.17 \frac{K^2\text{-FT.}}{IN^2} \times 12\ IN/FT}{2(30 \times 10^3\ K/IN^2)} = 0.5025 \times 10^{-2}\ IN.\text{-}K$$

$$\Delta = 1.005 \times 10^{-2}\ IN./K$$

$$K = \frac{1}{\Delta} = \frac{1}{1.005 \times 10^{-2}} = 99.5\ K/IN.$$

$$\therefore T_a = .32 \sqrt{\frac{W}{K}} = .32 \sqrt{\frac{10}{99.5}} = .102\ SEC. \quad (10\text{-}1)$$

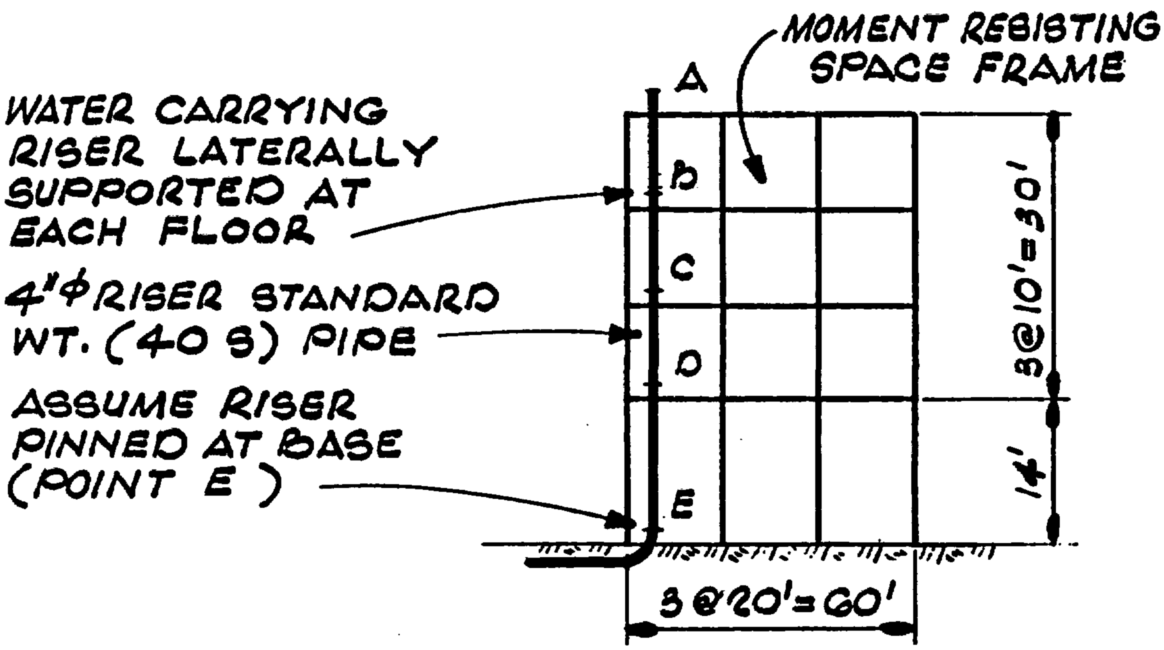
T_a (EQUIPMENT PERIOD) = 0.102 > 0.05 SEC.
SUPPORT IS NOT RIGID (PARA. 10-3)
DESIGN AS FLEXIBLY MOUNTED (PARA. 10-4)

T (BLDG. PERIOD) IS CALCULATED TO BE 0.31 SEC.
REFER TO PARA. 10-4c(1)
T_a/T = 0.102/0.31 = 0.33 AND T < 0.5 SEC.
FIND A_p FROM FIGURE 10-3a

$$A_p = 1 + \left(\frac{0.33 - 0.10}{0.80 - 0.10}\right)(5.0 - 1.0) = 2.31$$

$F_p = ZIA_pC_pW_p$, (PARA. 10-4e, FORMULA 10-2)
WHERE $Z = 3/8$ (TABLE 3-1)
$I = 1.0$ (TABLE 3-2)
$C_p = 0.30$ (TABLE 3-4)

$F_p = 3/8 \times 1.0 \times 2.31 \times 0.30 W_p = 0.26 W_p$
$F_p = 0.26 \times 10$ = <u>2.6 KIPS/TRUSS</u>



GIVEN: RISER AS SHOWN IN MULTI-STORY BUILDING. SEISMIC ZONE 4
ESSENTIAL FACILITY BUT THE RISER IS NOT RELATED TO FIRE PROTECTION

REQUIRED: FIND SEISMIC FORCE AT EACH LATERAL RISER SUPPORT.

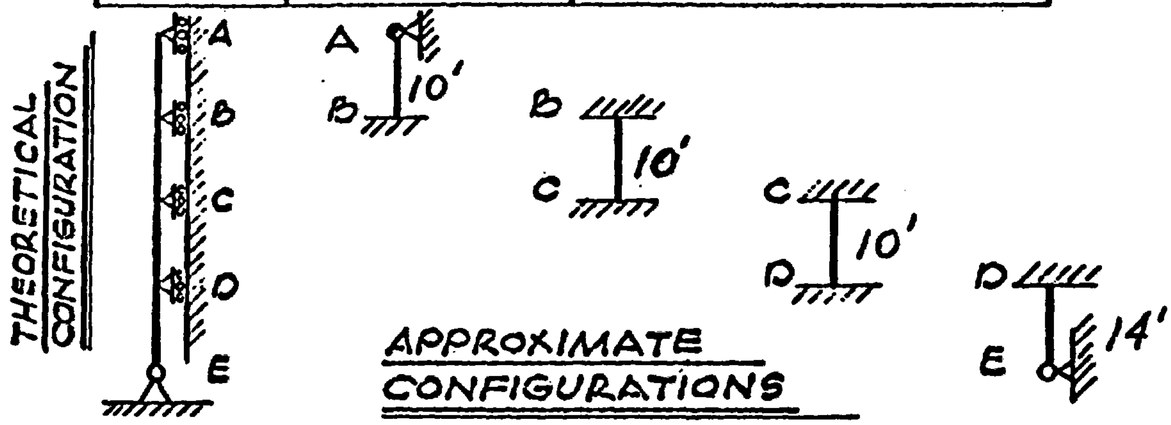
SOLUTION: AN APPROXIMATE SOLUTION WILL BE MADE.
FIRST INVESTIGATE THE ALLOWABLE SPAN FOR 4"φ (40 S) PIPE, THEN APPLY SEISMIC LOADING TO RISER.

1. IF PIPING SYSTEM IS RIGID
$F_p = ZIC_pW_p$ [PARA. 10-7C(2)]

2. IF PIPING SYSTEM IS NOT RIGID
$F_p = ZIA_pC_pW_p$ [PARA. 10-7C(3) & (4)]

Design Example F-9 | 1 of 2 | Water Riser

PIPE	APPROXIMATE END COND.	MAXIMUM RIGID SPANS (FIG. 10-4, 10-5 & 10-6)
A B	FIXED - PINNED	14'-6"
B C	FIXED - FIXED	17'-8"
C D	FIXED - FIXED	17'-8"
D E	FIXED - PINNED	14'-6"



APPROXIMATE CONFIGURATIONS

PIPE SPANS ARE SHORTER THAN MAXIMUM RIGID SPAN LIMIT; ∴ $F_p = Z\,I\,C_p\,W_p$ APPLIES.
$Z = 1.0$ (TABLE 3-1) $I = 1.5$ (TABLE 3-2); $C_p = 0.30$
W_p = (WT. OF PIPE + CONTENTS) = (10.8 + 5.5) LB/FT × LENGTH
$F_p = 1.0 \times 1.5 \times 0.30\ W_p = 0.45\ W_p = 7.3$ LB/FT.

POINT	APPROXIMATE TRIBUTARY LENGTH (FT.)	APPROXIMATE CONNECTION LOAD (LBS)
A	5.0	37
B	10.0	73
C	10.0	73
D	12.0	88
E	7.0	51

APPENDIX G
STRUCTURES OTHER THAN BUILDINGS

G-1. Purpose and scope. The design examples in this appendix are to illustrate principles, factors and concepts involved in seismic design. These are not mandatory; and other equivalent methods, materials or details complying with this manual and applicable agency guide specifications may be used.

G-2. Design examples:

Design Example	Description
G-1	*Elevated Tank (Braced Frame):* Four-legged, diagonal braced tower.
G-2	*Vertical Tank (On Ground):* Vertical water tank supported directly by the ground.
G-3	*Horizontal Tank (On Ground):* Typical horizontal tank supported on saddles.
G-4	*Pole-Mounted Transformer:* See Appendix F, Design Example F-2.
G-5	*Tower-Mounted Equipment:* See Appendix F, Design Example F-3.

DESIGN EXAMPLE: G-1

ELEVATED TANK (BRACED FRAME):

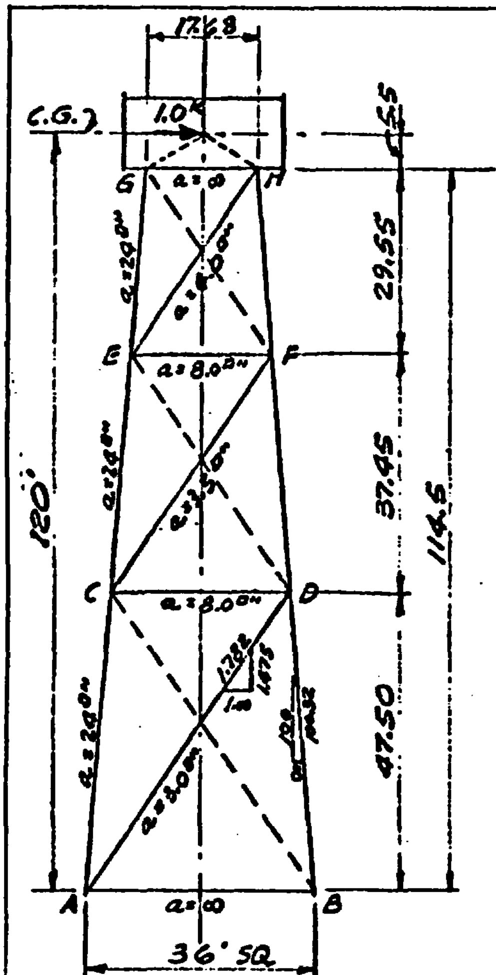
Description of Structure. A 100,000 gallon steel water tank on top of a 114.5 foot high steel braced frame.

Lateral Loads.

$V = ZIKCSW$ (3-1)

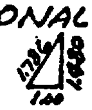
where $Z = 3/4$ (Zone 3, Table 3-1)
$I = 1.0$ (Table 3-2)
$K = 2.5$ (Table 3-3 and para. 11-3)
C and S are dependent on periods T and T_s

$T_s = 0.8$ sec (determined from a geotechnical investigation, para 4-3f)
$T = 1.46$ sec (calculated on sheet 2 of 2 of this example)
$C = 1/15\sqrt{T} = 1/15\sqrt{1.45} = 0.055$ (3-2)
$T/T_s = 1.45/0.8 = 1.81$
$S = 1.2 + 0.6(T/T_s) - 0.3(T/T_s)^2 = 1.30$ (3-4A)
$KC = 2.5\ (0.055) = 0.14 > 0.12$ (o.k., para 11-3)
$V = 3/4 \times 1.0 \times 2.5 \times 0.055 \times 1.30\ W = 0.134\ W$



ELEVATION

SLOPE OF DIAGONALS IN PLANE OF SIDE



DEFLECTION Δ*

MEMBER	LENGTH	AREA	u	u^2L/A
A B	36.0	∞	+.134	.0
A D	57.5'	3.0□"	+.474	4.30
A C	47.8'	24.0	+1.283	3.27
B D	47.8'	24.0	-1.678	5.60
C D	28.4'	8.0	-.296	.31
C F	45.3'	3.5	+.598	4.62
C E	37.7'	24.0	+.787	.97
D F	37.7'	24.0	-1.283	2.58
E F	22.4'	8.0	-.379	.39
E H	35.8'	4.0	+.758	5.14
E G	29.7'	24.0	+.156	.03
F H	29.7'	24.0	-.787	.77
G H	17.68'	∞	-.487	0
				27.98

100,000 GALLON WATER TANK

WEIGHT OF WATER	833^K
STEEL TANK (EST)	87
WATER + TANK	$920^K = W$

NEGLECT WT. OF TOWER

ASSUME BRACES CARRY TENSION ONLY.

COMPUTE THE PERIOD OF THE STRUCTURE TO DETERMINE COEFFICIENTS C AND S

$$T = 0.32\sqrt{\frac{W}{K}} \qquad (10-1)$$

$W = 920^K$

K = SPRING CONSTANT (KIPS/INCH)

IF A 1.0^K LATERAL LOAD IS APPLIED AT THE TANK C.G.,

$$K = \frac{1.0}{\Delta}$$

WHERE Δ = LATERAL DEFLECTION OF TANK DUE TO 1^K LOAD.

$$T = 0.32\sqrt{W \times \Delta}$$

$$\Delta = \frac{2^{*} \times 27.98 \times 12}{30{,}000} = 0.0224\ \text{IN}$$

FOR 1^K LOAD ON TOWER (0.5^K EACH SIDE)

$$T = 0.32\sqrt{920 \times 0.0224}$$

$$= 1.45\ \text{SEC}$$

* 0.5^K APPLIED TO EACH SIDE OF TOWER. FOR 1.0^K ON THE WHOLE TOWER:

$$\Sigma \frac{u^2L}{A} = 2 \times 27.98$$

$$\Delta = \Sigma \frac{u^2L}{AE}\ \text{IN/KIP}$$

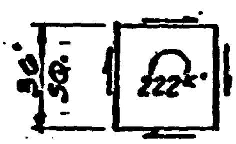
V = 0.134 W (SHEET 1 OF 1)
= 0.134 x 920 = 123.3 KIPS.

STRESS IN MEMBERS FOR LOAD APPLIED PARALLEL TO MAJOR AXIS. $V = 123.3^k$

MEMBER		DIRECT LOAD STRESS	ECCEN. LOAD STRESS	TOTAL STRESS	UNIT STRESS
AB	+.134 x 123.3^k	+ 16.5^k	+ 0.8^k	+ 17.3^k	—
AD	+.474	+ 58.5	+ 2.9	+ 61.4	20.5 k/□"
AC	+1.283	+158.2	0.	+158.2	6.6
BD	-1.678	-207	0.	-207	8.63
CD	-.296	- 36.5	- 1.8	- 38.3	4.79
CF	+.598	+ 73.7	+ 3.7	+ 77.4	22.1
CE	+.787	+ 97.1	0.	+ 97.1	4.05
DF	-1.283	-158.2	0.	-158.2	6.6
EF	-.374	- 46.2	- 2.3	- 48.5	6.06
EH	+.758	+ 93.4'	+ 4.7	+ 98.1	24.5
EG	+.156	+ 19.2'	0.	+ 19.2	0.80
FH	-.787	- 97.1	0.	- 97.1	4.05
GH	-.487	- 60.1	- 3.0	- 63.1	—

STRESSES DUE TO 5% ECCENTRICITY

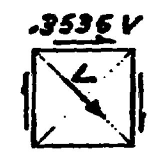
$M_T = .05 \times 36 \times 123.3 = 222'^k$



SHEAR ON EA. OF 4 SIDES $= \frac{222}{4 \times 18} = 3.08^k$

STRESS IN WEB MEMBERS $= \frac{3.08}{(123.3/2)} \times$ (DIRECT LOAD STRESS)
STRESS IN COLUMNS = 0

CHECK COLUMN FORCES AND UPLIFT FOR LOAD APPLIED AT 45° TO MAJOR AXIS OF TOWER



$P = \frac{123.3 \times 120}{1.414 \times 36} \times 1.007 = \pm\ 293$ KIPS

(NOTE: FORCE IN BD $\times \sqrt{2}$ = 207 x 1.414 = 293)

GRAVITY FORCE ON COLUMNS = $920^k \div 4$ = -230 KIPS

COLUMN DESIGN: -293 - 230 = -523 KIPS (COMPR.)

UPLIFT: +293 - 0.9* (230) = 86 KIPS (UPLIFT)

DESIGN ANCHOR BOLTS AND FOUNDATION FOR 86 KIPS UPLIFT FORCE

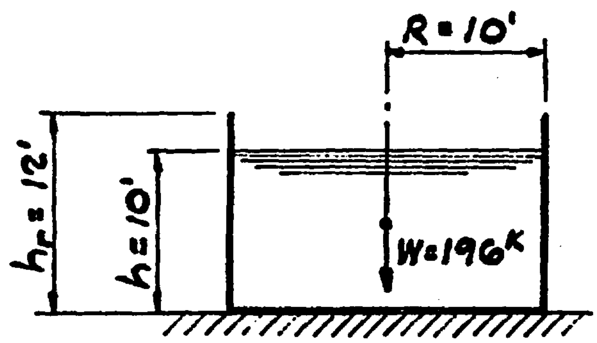
*REFER TO PARA. 3-3(A)4, 3-3(J)2c, AND 4-4c (2)

DESIGN EXAMPLE: G-2

VERTICAL TANK (ON GROUND)

Description of Structure. A cylindrical water tank on grade with a radius of 10 feet (R = 10), a height of 12 feet (h_T = 12), and a water depth of 10 feet (h = 10). The tank is located in Seismic Zone 4, I = 1.0, and T_S is unknown. The weight of the tank is 20 kips.

Required. The period of the sloshing water, the maximum vertical displacement of the water (d_{max}), and the design seismic forces. Refer to Chapter 11, paragraph 11-4.



REFER TO FIGURES 11-1 AND 11-2 FOR SEISMIC FORCE DISTRIBUTION

GENERAL

Z = 1.0, SEISMIC ZONE 4 (TABLE 3-1)
I = 1.0 (TABLE 3-2)
K = 2.0 (TABLE 3-3 AND PARA. 11-4)
C = $1/15\sqrt{T} \leq 0.12$ (3-2)
S = MAXIMUM VALUE (T_S NOT KNOWN)
α = h/R = 10.0/10.0 = 1.0
W (WATER) = $\pi(10)^2(10)(0.0624) = 196^K$
W_r (ROOF) = 0 (NO ROOF)
W_W (TANK WALLS) = 20^K

RIGID BODY FORCES [PARA. 11-4a(1)]

$$V_{RB} = ZIKCS\,(W_r + W_W + W_I) \quad (11\text{-}1)$$

$$CS = 0.14 \text{ (TABLE 4-3, } T < 0.3 \text{ SEC.)}$$

$$ZIKCS = 1 \times 1 \times 2 \times 0.14 = 0.28$$

$$W_I = 0.54W \quad \text{(TABLE 11-1)}$$
$$= 0.54 \times 196 = 106^K$$

$$V_{RB} = 0.28\,(0 + 20 + 106) = \underline{35.3^K}$$

$$h_I = 0.38h \quad \text{(TABLE 11-2)}$$
$$= 0.38 \times 10 = 3.8 \text{ FT.}$$

$$h_I' = 0.78h \quad \text{(TABLE 11-2)}$$
$$= 0.78 \times 10 = 7.8 \text{ FT.}$$

$$M_{RB}\text{(TANK SHELL)} = ZIKCS\left[W_r h_r + W_W \bar{h}_W + W_I h_I\right] \quad (11\text{-}2)$$
$$= 0.28\left[0 + 20\left(\tfrac{12}{2}\right) + 106(3.8)\right]$$
$$= \underline{146^{K\text{-}FT}}$$

$$M_{RB}\text{(BELOW BASE)} = 0.28\left[0 + 20\left(\tfrac{12}{2}\right) + 106(7.8)\right]$$
$$= \underline{265^{K\text{-}FT}}$$

SLOSHING WATER FORCE [PARA. 11-4a (2)]

PERIOD, $T = k_T\sqrt{h}$ (11-4)

$k_T = 0.84$ (TABLE 11-3)

$T = 0.84\sqrt{10} = \underline{\underline{2.66 \text{ SEC.}}}$

$V_{SL} = ZIKCSW_C$ (11-3)

$C = 1/15\sqrt{2.66} = 0.041$

$S = 1.5$ (MAXIMUM VALUE)

$ZIKCS = 1 \times 1 \times 2 \times 0.041 \times 1.5 = 0.123$

$W_C = 0.43W$ (TABLE 11-1)

$= 0.43 \times 196 = 84.3^K$

$V_{SL} = 0.123 \times 84.3 = \underline{10.4^K}$

$h_c = 0.60h = 0.60 \times 10 = 6.0$ FT. (TABLE 11-2)

$h_c' = 0.79h = 0.79 \times 10 = 7.9$ FT.

M_{SL} (TANK SHELL) $= ZIKCS\,W_C h_C$ (11-5)

$= 0.123 \times 84.3 \times 6.0$

$= \underline{62.2^{K\text{-}FT}}$

M_{SL} (BELOW BASE) $= 0.123 \times 84.3 \times 7.9$

$= \underline{81.9^{K\text{-}FT}}$

HEIGHT OF SLOSHING WATER

$$d_{MAX} = \left[\frac{0.75\,(ZIKCS)}{1-k_d(ZIKCS)}\right]R \qquad (11\text{-}6)$$

$$= \left[\frac{0.75\,(0.123)}{1-(1.75)(0.123)}\right]10.0 \qquad (k_d \text{ FROM TABLE 11-4})$$

$$= \underline{\underline{1.17 \text{ FT.}}} \quad (\text{LESS THAN } h_r - h = 2 \text{ FT, OK})$$

TOTAL DESIGN FORCES [PARA. 11-4a(3)]

$$V_{TOTAL} = \sqrt{V_{RB}^2 + V_{SL}^2} \qquad (11\text{-}8)$$

$$= \sqrt{(35.3)^2 + (10.4)^2} = \underline{\underline{36.8^K}}$$

$$M_{TOTAL} = \sqrt{M_{RB}^2 + M_{SL}^2} \qquad (11\text{-}9)$$

$$\text{FOR TANK SHELL} = \sqrt{146^2 + 62.2^2} = \underline{\underline{158^{K\cdot FT}}}$$

$$\text{FOR BELOW BASE} = \sqrt{265^2 + 81.9^2} = \underline{\underline{277^{K\cdot FT}}}$$

DESIGN EXAMPLE: G-3

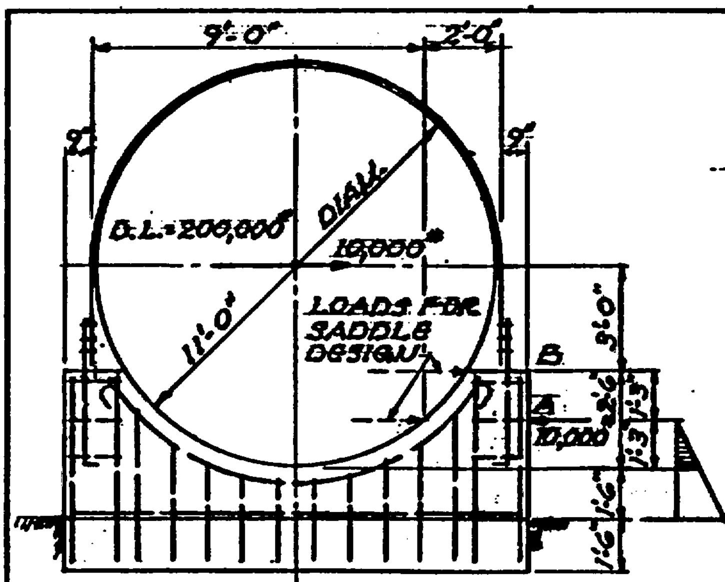
HORIZONTAL TANK (ON GROUND):

Description of Structure. A 20,000 gallon steel tank in concrete saddles on a concrete slab on grade. Seismic Zone 2, I = 1.0, T_s = 2.5 sec.

Lateral Loads:

$V = ZIKCSW$

where Z = 3/8, I = 1.0, K = 2.0, T_s = 2.5, assume $T < 0.3$ sec.
CS = 0.133 (Section 4-3, Table 4-3)
W = Weight of tank plus contents.
$V = 3/8\,(1.0)\,(2.0)\,(0.133)\,W$
$= 0.10\,W$



20,000 GALLON TANK
11'-0" DIAM. x 28'-0" LONG
WEIGHT TANK PLUS
CONTENTS 200,000 LBS.
SEISMIC LATERAL FORCE
V = ZIKCSW = 0.10 W
= 0.10 x 200,000
= 20,000 LBS.
OR 10,000 LB. EA. SADDLE

STRAP DESIGN:

FOR THE PURPOSE OF THIS EXAMPLE ASSUME THE REACTION IS AT LEVEL "A" AND NEGLECT WEIGHT OF TANK AND CONTENTS.

$M = 10000 \times 4.25 = 42,500'^{\#}$

$STRESS = \frac{42,500}{9.0} = 4,720^{\#}$ IN STRAP

SADDLE DESIGN

FOR REINFORCEMENT ASSUME THE LOAD ON THE PIER TO BE APPLIED AT LEVEL "B"

MOMENT WITH LOAD APPLIED AT LEVEL B

$M = 10,000 \times 2.5 = 25,000'^{\#}$ DESIGN REINF. TO RESIST THIS BENDING MOMENT IN ACCORDANCE WITH STANDARD PROCEDURE.

BASE DESIGN

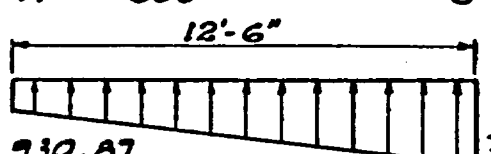
TOTAL O.T.M. $= 20,000 \times 8.5 = 170,000'^{\#}$

BASE 12'-6" x 24'-0" $A = 12.5 \times 24 = 300^{\square'}$

SECTION MODULUS $S = \frac{24 \times (12.5)^2}{6} = 626$

EA. SADDLE $= 5880^{\#} \times 2 = 11,760^{\#}$
BASE WEIGHT $= 225 \times 300 = 67,500^{\#}$
+200,000
279,260# TOTAL WEIGHT

$\frac{P}{A} = \frac{279,260}{300} = 930.87 \quad \frac{M}{S} = \frac{170,000}{626} = 271$

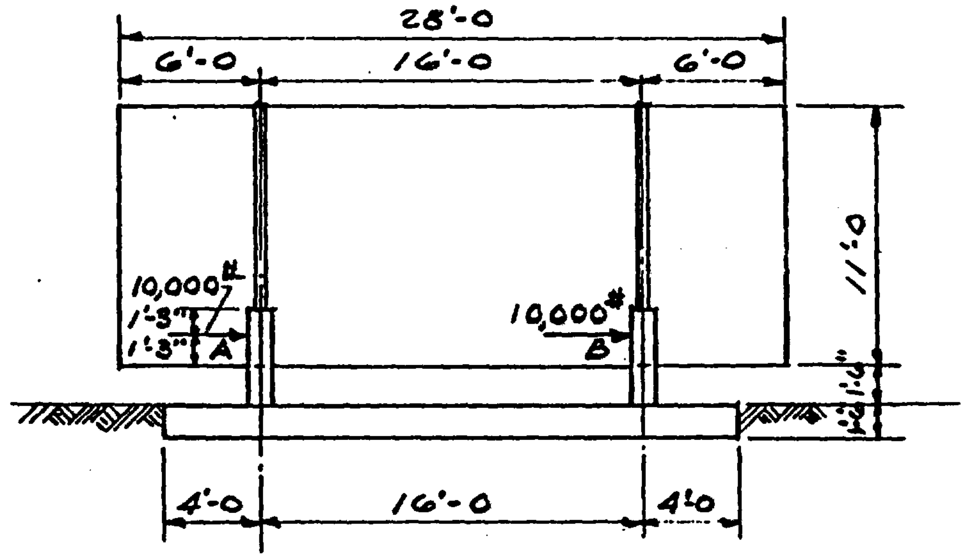


930.87
+271.0
1201.87 #/□'

930.87
-271.0
659.87#/□'

RESULTANT IS IN MIDDLE THIRD

Design Example G-3	2 of 3	Horizontal Tank on Ground



OVERTURNING ON SUPPORT IS NEGLIGIBLE AND IS NOT INCLUDED IN THIS CALCULATION

SADDLE DESIGN

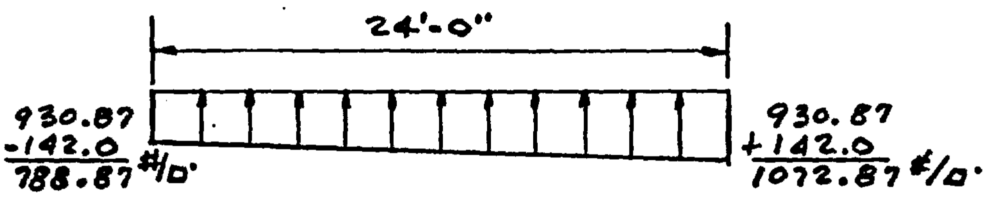
$M_A = M_B$ ABOUT BASE OF TANK = 10,000 × 1.25 = 12,500 '#
ABOUT FOOTING = 10,000 × 2.75 = 27,500 '#
DESIGN REINF. TO RESIST THESE BENDING MOMENTS IN ACCORDANCE WITH STANDARD PROCEDURE

BASE DESIGN

DESIGN REINF. IN FOOTING IN ACCORDANCE WITH STANDARD PROCEDURE TO RESIST SADDLE M = 27,500 '#
TOTAL O.T.M. = 20,000 × 8.5 = 170,000 '#

SECTION MODULUS $S = \frac{12.5K(24)^2}{6} = 1200$

$\frac{P}{A} = 930.87$ (FROM SHEET 2 OF 3) $\frac{M}{S} = \frac{170{,}000}{1200} = 142$



RESULTANT IS IN MIDDLE THIRD
DESIGN FOOTING FOR SOIL PRESSURES SHOWN IN ACCORDANCE WITH STANDARD PROCEDURE.

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