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A METHODOLOGY FOR SEISMIC DESIGN AND CONSTRUCTION OF SINGLE-FAMILY DWELLINGS

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by

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for

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Division of Energy, Building Technology and Standards

NOTICE

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ABSTRACT

This report presents the results of an in-depth effort to develop design and construction practices for single-family residences that minimize the potential economic loss and the life-loss risk associated with earthquakes. The report:

- Discusses the ways structures behave when subjected to seismic forces;
- Sets forth suggested design criteria for conventional layouts of dwellings constructed of conventional materials;
- Presents construction details that do not require the designer to perform analytical calculations;
- Suggests procedures for efficient plan-checking; and
- Presents recommendations including details and schedules for use in the field by construction personnel and building inspectors.

† † † † †

PREFACE

Observations and investigations of earthquake damage have revealed that contemporary empirical design and construction practices for single-family residences have not in all cases proven adequate. Recognizing this, the Department of Housing and Urban Development engaged the Applied Technology Council, San Francisco, to review available damage information as well as existing design and construction requirements for the purpose of developing a methodology for improved seismic design and construction for single-family dwellings, thereby reducing future siesmic losses and life-loss risks.

The Applied Technology Council (ATC) is a nonprofit corporation, established by the Structural Engineers Association of California, and has as its goal the protection and enhancement of the public welfare through the advancement of technological developments for application in daily practice in the fields of structural design and construction. In particular, the objectives of ATC are to gather, coordinate, and disseminate technical data in order that it may be made available for use by engineers, other design professionals, code-enforcing officials, and builders.

The ATC organization fills a unique need for the design professionals, providing a means to assemble expert talent to solve special problems as they arise. ATC does not have a staff of researchers. Instead, all work is performed under subcontracts or agreements with qualified individuals, institutions or firms. The system makes it possible to bring current state-of-the-art techniques and methodology together with seasoned experience of practicing professionals.

To develop the information needed to improve seismic resistive construction practices for single-family residences, ATC engaged a structural engineer subcontractor well-experienced in residential construction and earthquake engineering, plus an advisory panel consisting of prominent structural engineers and a representative of the Building Industry Association (San Fernando Valley Chapter).

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Although the work reported in this publication was performed under the general supervision of ATC, the findings, conclusions, and/or recommendations are those of the consultants and subcontractor listed. The manuscript and details were prepared by Ralph W. Goers SE and reviewed by the Advisory Panel. Architectural consultation as well as preparation of the plans and elevations for the example homes were provided by Donald G. Park AIA of Benton/Park/Candreva, Associates, Los Angeles. Editorial consultation and publication of the report were provided by William Giles & Associates, San Francisco.

† † † † †

EXECUTIVE SUMMARY

On the morning of February 9, 1971, California's San Fernando Valley shook. Within hours the world knew that an earthquake, only moderately strong, had struck a major population center and had left in its wake death and damage to virtually all types of structures.

On the positive side, this earthquake, the first to strike a large and relatively new metropolitan area, provided an opportunity to investigate contemporary single-family-dwelling construction practices. Several studies of damage to residential-type buildings were conducted and the relative merits and shortcomings of various aspects of construction practices were identified. A principal conclusion of these damage studies was that contemporary empirical design and construction requirements for one-story wood-frame dwellings provided adequate life-safety protection, but that occupants of two-story, split-level and other types of unusual configurations of wood-frame dwellings do not have the same life-safety protection. It was also an apparent conclusion that dollar losses for dwellings subjected to earthquakes could be reduced significantly if the dwellings were provided with additional bracing to resist lateral forces.

Based on the findings from the damage studies, the effort reported here was undertaken for the purpose of developing a methodology for improving the seismic resistance of single-family residences.

The principal tasks in this total project included: (1) a review of the damage to single-family residences caused by earthquakes in the United States, with emphasis on the damage caused by the San Fernando earthquake; (2) a review of HUD/FHA MPS and MAP plus appropriate building code earthquake requirements applicable to dwellings; (3) development of a seismic design and construction methodology for single-family residences; and (4) an evaluation of the cost impact of these seismic design criteria. The seismic design and construction methodology is presented in this report; the information assembled for the other tasks is available in three other reports.

This report is divided into five parts:

- I Introduction
- II Basic Seismic Analysis for Single-Family Residences
- III Seismic Design Methodology
- IV Construction Details
- V Guidelines for Plan-Checkers and Inspectors

In addition to stating purpose, scope, limitations, etc., the Introduction also points up the need for improved design criteria and construction practices with a general description and summary of damage to single-family residences observed after the San Fernando earthquake.

To develop a structure to resist earthquakes, it is essential that the home designer understand the way buildings act when subjected to earthquakes. Part II of this report includes, first, a chapter on the principles of seismic design, describing the way earthquakes impart forces to a structure, identifying the most important building components and connections that act to resist these forces, and outlining the steps in earthquake-resistive design. The remainder of Part II consists of chapters giving detailed seismic-resistive design examples and commentary based upon the methodology of the report.

Part III contains the methodology for seismic-resistive design of single-family residences. The criteria include consideration of structural (wood-frame and masonry) and nonstructural components of houses. Specifically, the criteria include consideration of all aspects of structural design necessary to achieve a structure capable of providing life-safety during earthquakes. The design criteria are presented using nontechnical terminology wherever possible.

Construction details illustrating the procedures required to achieve seismic-resistive performance of residences are presented in Part IV. Although the details presented can be used directly for a given design, they are not intended to require unnecessary changes in local standardized procedures. If local standards meet the intent of the details, the specific detail shown need not be followed. In addition to presenting modifications to existing construction practices, the report includes new details to achieve greater strength and/or safety for items that have proven to be inadequate in earthquakes. These include, for example, special garage front wall details, a cabinet detail, a water heater detail, and criteria outlined, but not detailed, for other mechanical equipment, for cutting and notching of studs, etc.

Part V, Guidelines for Plan-Checkers and Inspectors, includes two chapters of commentary on the design criteria and construction details, commentary that is specifically oriented to the checking of plans and to field inspection. The commentary suggests efficient and expeditious plan-checking and inspection, highlighting the most common omissions and inadequacies of contemporary seismic-resistive design and construction of single-family residences.

† † † † †

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PART I INTRODUCTION

CHAPTER I–1 INTRODUCTION

Although the state-of-the-art in seismic resistive design is still in its infancy, much has been learned from recent earthquakes in Anchorage, Alaska (1964), Santa Rosa, California (1969) and, primarily, from the havoc strewn in the wake of the relatively moderate quake on the morning of February 9, 1971 in California's San Fernando Valley. Estimates of damage to single-family dwellings in the San Fernando earthquake ranged from $58 million to $114.4 million. The damage to residences — mostly of modern design and construction — was of greater dollar value than damage to any other building category in the private sector. The goal of this publication is to demonstrate improved residential design and construction methods to mitigate such hazards to life and property.

Purpose

The purpose of this publication is to illustrate practical design methods and construction details with the intent of reducing the structural damage inflicted on residential dwellings by earthquakes.

The methodology given in this report includes consideration of the engineering principles involved in seismic design. The report is developed, however, for use by housing designers and others familiar with construction but not necessarily familiar with structural engineering.

Types of Design and Construction Materials

The design configurations discussed are representative of the vast majority of houses constructed of wood frame or masonry in the United States today: One-story houses of simple floor plan, two-story houses with attached garages, and split-level houses of various types.

Design. Contemporary architectural design of houses has become much more sophisticated than simply "four walls and a roof." The demands for larger rooms, more glass, cathedral or open-beam ceilings, split levels and split entries have resulted in residential structures with either weaknesses inherent to the design or with lessened rigidity so far as earthquake resistance is concerned. The design requirements in current building codes are not definitive enough for many modern home configurations. On the other hand, to require every home to be structurally engineered would be unrealistically costly. With the advent of the inexpensive electronic calculator, however, a middle ground is available — improved seismic resistive design is possible with only relatively simple calculations. While they will not prevent all residential damage, the improved designs should greatly reduce the effects of strong seismic disturbances on house construction.

It is emphasized that this report does not attempt to cover specialty designs requiring complicated engineering analyses, nor does it cover problems encountered in the design of houses with special grading conditions. For non-conventional designs or design conditions, the designer is referred to professional engineering consultants.

Construction Materials. The report considers those construction materials used most commonly for residences, including wood framing with siding, wood frame and masonry veneer, wood frame and stucco, and brick or block masonry. Manufactured or prefabricated housing and other systems using innovative materials other than the standard concrete foundation, wood-frame or masonry exterior walls and "standard" wood framing methods are not included because of infrequency of their use or the lack of standardization of materials and construction methods. Finish materials used as shear-resisting materials include all the broad categories for which racking tests are available. Tests of other individual materials and other methods of fastening have been made by individual manufacturers and have received acceptance by many code authorities.

Basis of Methodology

In general, the requirements in this report are in accordance with pertinent provisions of the Uniform Building Code (UBC) produced by the International Conference of Building Officials (ICBO) and with the "Recommended Lateral Force Requirements and Commentary" published by the Structural Engineers Association of California (SEAOC). Additional requirements have been included to cover certain aspects of design and detailing. For example, conventional provisions for light-frame construction include bracing requirements and, if met, no structural design is required for single-family wood-frame dwellings. This report is based on the premise that additional bracing requirements are needed for many modern house configurations. To require detailed engineering analyses would be prohibitive for house construction. Seismic coefficients used in this publication are the same as the coefficients used in the ICBO (UBC, 1973 Edition) and SEAOC publications.

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CONSTRUCTION DETAILS

See Table 4.1, Page IV–3, for List of Construction Details

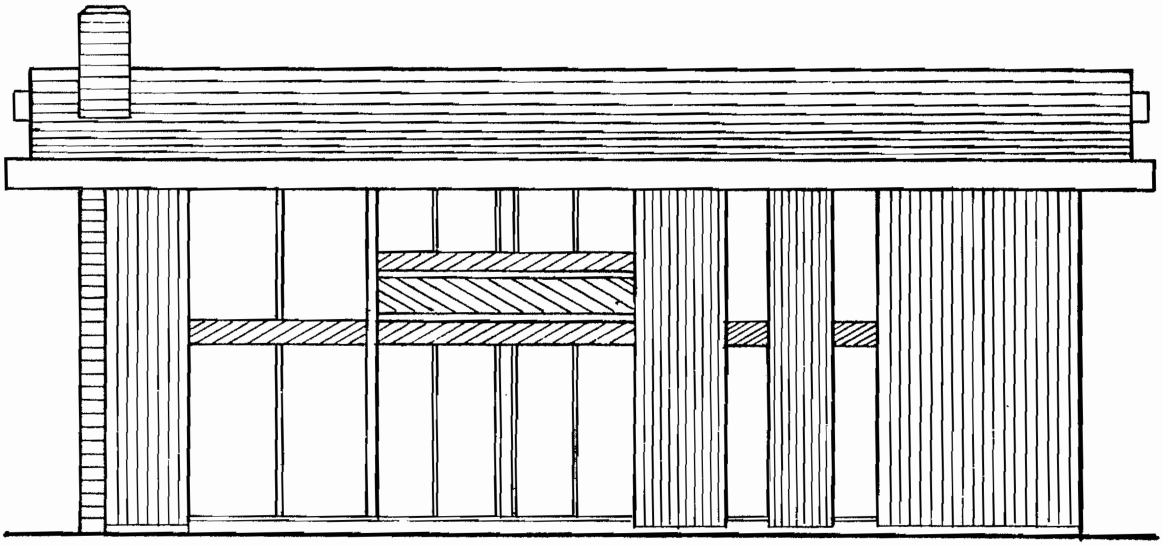

FIGURE 1.1.

Limitations on Design

Although many types of home designs are provided for in the publication, the designer must recognize that some more complicated designs will necessitate engineering analysis. The following housing conditions, for example are not within the scope of this publication:

- Though a method is indicated in a subsequent chapter for the design of two-story-high shear walls, it is recommended that a designer not attempt houses with elevations similar to that shown in Figure 1.1. Such a house differs from the "normal" two-story house in that the glass between the walls is full height with only the floor framing intervening. This type of construction presents problems beyond the scope of the methodology in this report.

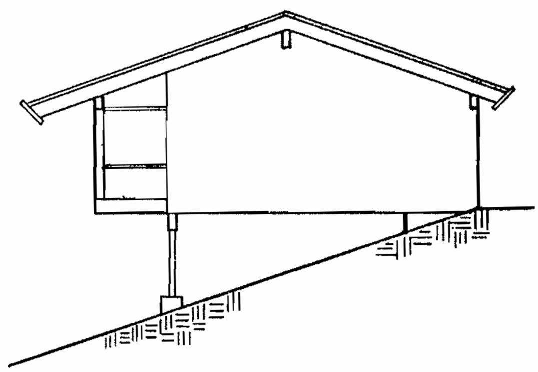

FIGURE 1.2(A)

- Of the two hillside homes illustrated in Figure 1.2, the "stilt" house in (A) must be engineered to achieve proper seismic resistance. The house in (B) can be designed according to provisions in the report, however, if the floor line at finish grade on the right is considered as a second floor. If no living space were provided below this floor line, the design would normally prove to be quite conservative. If such a design required shear-resisting materials other than those intended to be used as the finish material, economy would probably be achieved by having the house engineered.

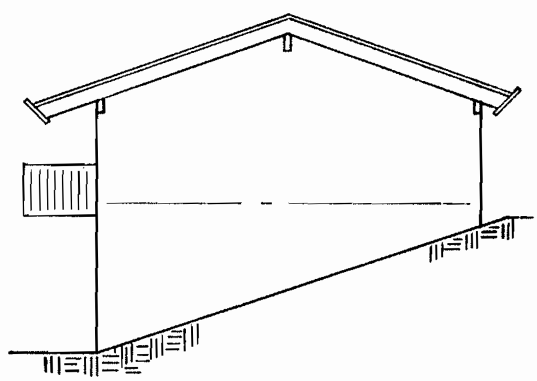

FIGURE 1.2(B)

New Concepts and Procedures

Several concepts are introduced in this report which are not currently covered in standard structural design procedures. They include tying together split levels and other components of houses with broken roof or floor diaphragms, tying down mechanical equipment, designs for short walls at the front of garages, fastening studs to sill plates with framing anchors, and providing larger design loads to some first-story interior walls in two-story construction. Although the methods and details provided are not unduly conservative, the details implementing the new concepts and procedures are intended to correct observed deficiencies and therefore require additional material or stronger connections.

Primary consideration has been given to construction costs in the establishment of the criteria for design. Procedures set forth for design and detailing come as close as possible to those that would be developed in a full engineering analysis, but are necessarily not always as precise. The suggested procedures are more realistic than current empirical practices, however, and some construction cost increase may be expected. Details presented are intended to achieve the largest increase in seismic structural resistance for the least incremental increase in cost.

Limitations on Use of Publication

This publication should not be considered as a relief for the home designer or builder from the responsibility of complying with local ordinances, building codes and regulations or other requirements of health and safety authorities. The Department of Housing and Urban Development does not assume the responsibility for enforcing or determining compliance with any local codes or regulations or make interpretations regarding their application in any specific instance.

Some generalized criteria are formulated in this report which, though they cannot anticipate every possible use or interpretation, are more accurate than current practices. It is emphasized, therefore, that the Department of Housing and Urban Development, its contractors and subcontractors represent the requirements and recommendations presented only as minimum standards without implying that all possible conditions reflecting seismic dangers in a large variety of houses will be met or that the details and methods presented will be fully adequate in all cases.

Organization and Coverage

This report is presented in five Parts as follows:

- **Part I's** second chapter illustrates with photographs and text examples of earthquake damage pointing to the need for revised design and construction of single-family dwellings.

- **Part II** discusses the ways structures act when subjected to seismic forces, introducing principles of seismic design for residences. It will serve as a primer for those who are unfamiliar with earthquake load analysis techniques, including commentary on the methods and assumptions presented in the report, plus design examples on the proper use of the seismic design principles. It is intended that Parts I and II are to be used as references only on the purposes of the report and the principles involved.

- **Part III** sets forth in a more formal manner the Design Methodology developed in Part II, recommending design criteria for conventional layouts of single-family dwellings constructed of conventional materials.

- **Part IV** completes the Design Methodology by presenting typical construction details plus commentary on the details.

- **Part V** presents suggested procedures for use of the report by plan-checkers and by construction personnel and inspectors.

† † † † †

CHAPTER I—2 EARTHQUAKE-CAUSED DAMAGE TO RESIDENCES

The San Fernando, California, earthquake struck at six in the morning when most people were at home and many still in bed. Figures 1.3A and 1.3B below illustrate the severe effect of the earthquake on a split-level home. The house separated at the intersection of its levels (Figure 1.3A) and the two-story portion collapsed (a close-up view of the intervening space in Figure 1.3B).

It is remarkable that the death and major injury rates among the occupants of the San Fernando area residences — modern construction — were so low, considering the damage shown in the photographs below and the subsequent photographs in this report.

FIGURE 1.3A. A split-level house, vibrated apart at the intersection of its levels by the San Fernando quake.

FIGURE 1.3B. Close-up of the intersection area.

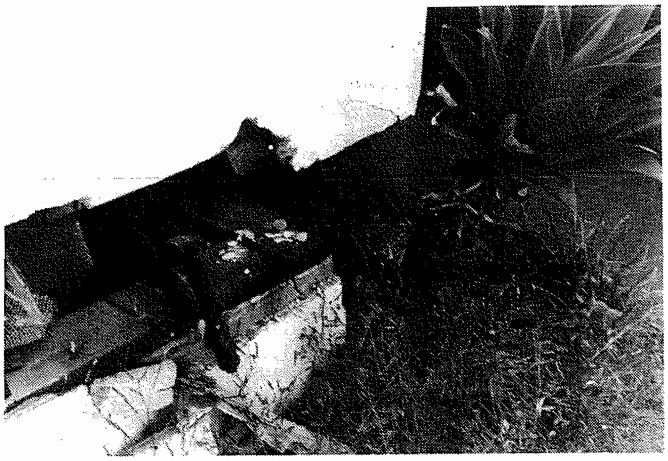

FIGURE 1.4A. A view of the front of a house subjected to the violence of earthquake faulting – the San Fernando faulting occurred directly beneath this house.

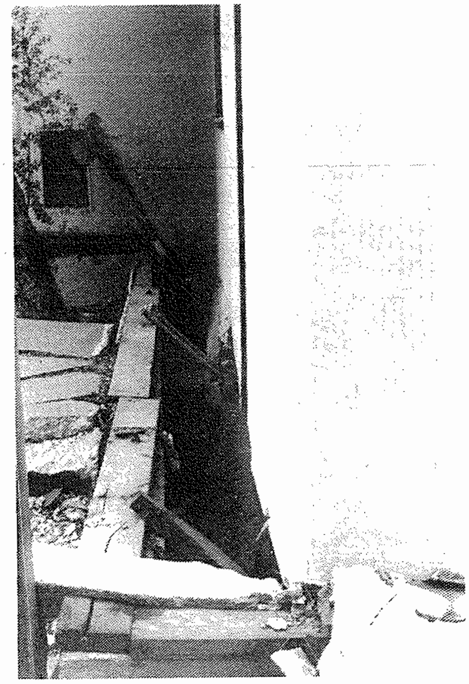

FIGURE 1.4B. A view of the rear of the same home.

Most of the damage in a San Fernando-type earthquake results from vibration, from landslide, or from severe ground distortion such as faulting.

Figures 1.4A and 1.4B show the front and rear of a house with a raised wood floor that was thrown off its foundation by the violent motion associated with faulting occurring directly beneath the house. Although the damages due to landslides and faulting are severe, they represent a very small percentage of total residential damage cost in most earthquakes. They may be averted by judicious avoidance of known faults and other perilous site conditions but they cannot be prevented by feasible structural design.

Vibration can, and often does, cause similar damage – compare the illustrations above with Figure 1.5 at the right.

Though frequently not as severe as landslide and faulting damage, but because it encompasses a much wider area – affecting virtually every structure within that area – earthquake vibration damage is the subject matter of this publication.

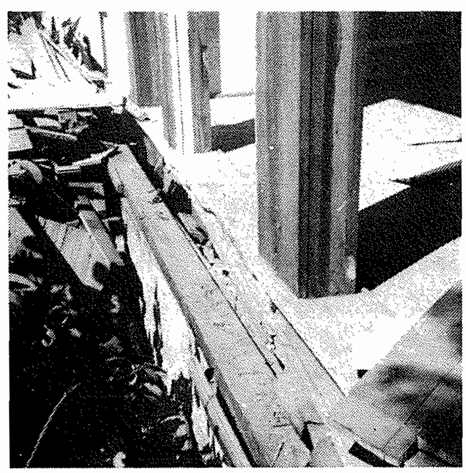

FIGURE 1.5. A general illustration of damage to modern residential construction that can be inflicted by the vibration forces generated by an earthquake.

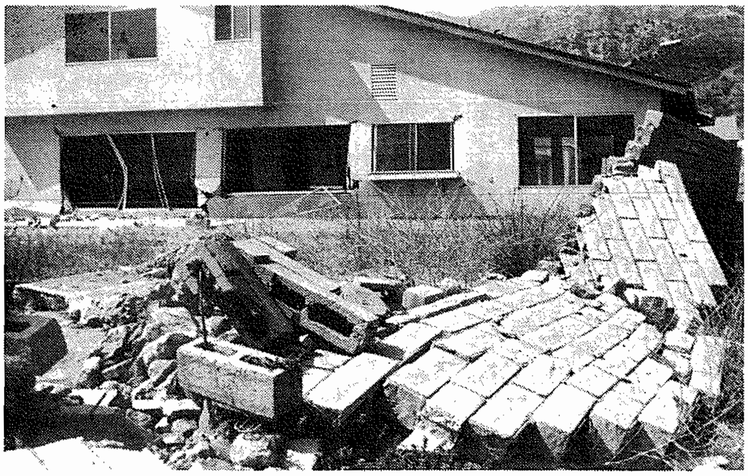

FIGURE 1.6A. A combination one- and two-story house that had relatively little shear wall at the rear.

FIGURE 1.6B. Interior view of damage to wall at the left in Figure 1.6A.

Figures 1.6A – 1.6D are views of a combination one- and two-story house that had relatively little wall at the rear of the first floor to resist horizontal forces. As described in later chapters, solid walls used to resist such forces are known as shear walls. Interior damage to the shear wall at the left of Figure 1.6A is shown in Figure 1.6B.

The shear wall at the front of the house (Figure 1.6C) failed in a different manner; it rotated about its lower right hand corner. One of the primary weaknesses in wood frame construction during the San Fernando quake proved to be the connection of studs to sill plates. The close-up (Figure 1.6D) of the rotated wall shows that the sole plate remained in place but the studs pulled free. The wall in Figures 1.6C and D is said to have failed in overturning.

(Chimneys are discussed in more detail later but note (Figure 1.6C) that the strap ties at the chimney held, at least in part due to the fact that the roof extended beyond the outside of the chimney; the entire fireplace nevertheless was destroyed.)

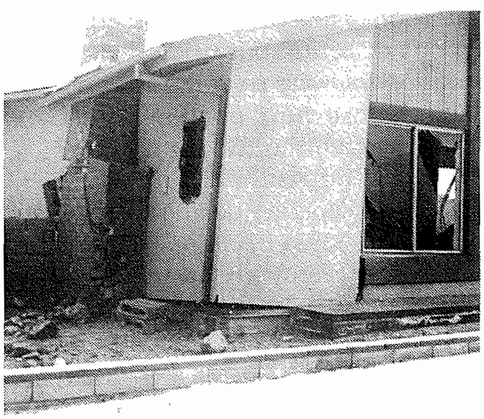

FIGURE 1.6C. A shear wall rotated about one of its corners.

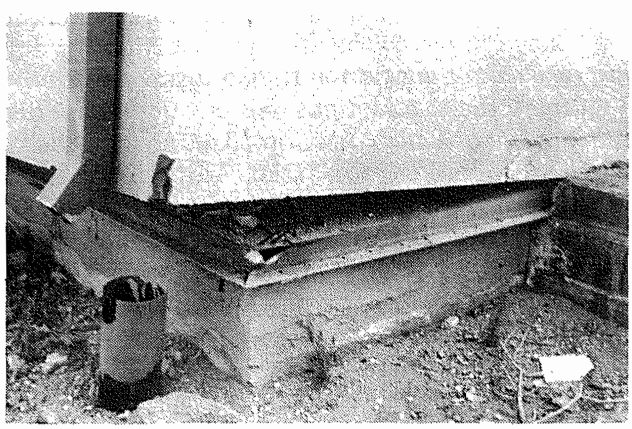

FIGURE 1.6D. Close-up of rotated wall shows the sole plate in place but studs pulled away.

FIGURES 1.7A – 1.7E.

Because there was virtually no resistive shear wall on either side of the garage doors, the second floor of this split-level house rotated in the horizontal plane, causing the "wall" in the front of Figure 1.7A to rotate to the left and the farther "wall" to rotate to the right. The wall between the garage and the family room became a fulcrum and the second floor and all above it moved back and forth as if on a sideways teeter-totter.

Figure 1.7B shows that although the wall at the rear of the house was "solid," it could not withstand the force of the rotating second floor and was pulled off its foundation.

Figures 1.7C and 1.7D show the separation between the two sections of the house.

Figure 1.7E shows the effect on the wall between the garage and the family room.

Photos by Jim Kronman

Figures 1.7A through 1.7E illustrate several types of failure. Although the house is split-level construction (the same model as shown in Figures 1.3A and B), the kinds of damage illustrated were encountered in other styles of houses as well. The problems started at the front of the garage (Figure 1.7A). Since virtually no wall was present on either side of the garage doors to provide resistance, the entire second floor of the house rotated in the horizontal plane, causing the wall on the side of the door closest in the picture to rotate to the left and the portion beyond to rotate to the right. The interior wall, between the garage and the family room behind, acted as a fulcrum for the sideways teeter-totter effect of the second floor and all above it. Although the wall at the rear of the house was solid it could not resist the rotation of the second floor and was pulled off its foundation. This is shown in Figures 1.7B and 1.7C. Note again that the sill plate is still connected to the foundation and that the wall failed at its point of weakness, the connection of the studs to the sill plate.

Once the rear wall failed, the only remaining wall at the first floor capable of resisting horizontal forces was the wall between the garage and the family room. The one-story portion of the house supported the two-story portion for motion in one direction but when the earthquake caused motion in the opposite direction, this interior wall was required to support all the force; it could not without significant distress and the two-story portion leaned away from the one-story portion. This created the separation between the two parts of the structure (Figures 1.7C and 1.7D). Note that there were no connections other than nails in tension supplied to resist the tendency for structures of this sort to separate during earthquakes. The shear wall between the garage and the family room is shown from the garage side in Figure 1.7E. Bear in mind that this lath and plaster wall was eventually asked to carry 100% of the two-story portion of the house in sidesway. It was incapable of resisting the lateral force and failed.

Occupants of the home shown in Figures 1.3A and B told interviewers that their house collapsed after failures of exactly the same type. They noted that the rear wall had moved out into the back yard before the second story collapsed during the first aftershock.

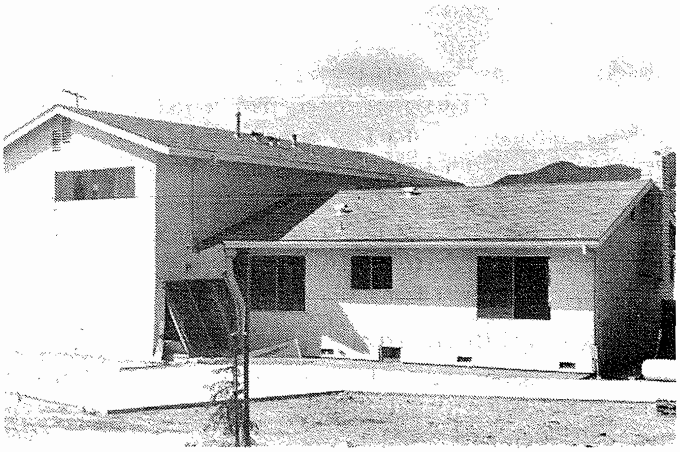

FIGURE 1.7B.

FIGURE 1.7C.

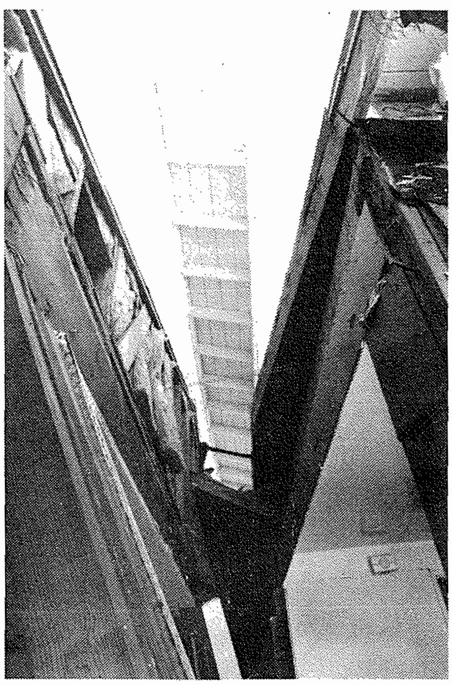

FIGURE 1.7D.

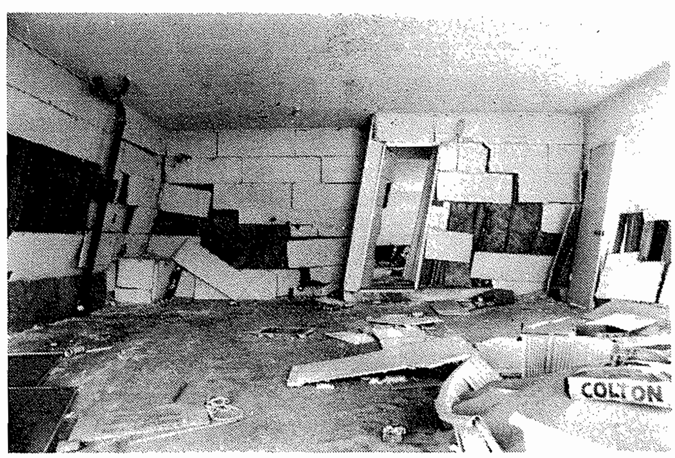

FIGURE 1.7E.

FIGURE 1.8A. Split-level victim of inadequate bracing at the front of the garage and the typical failure of "cripple" stud walls under the mid-level portion.

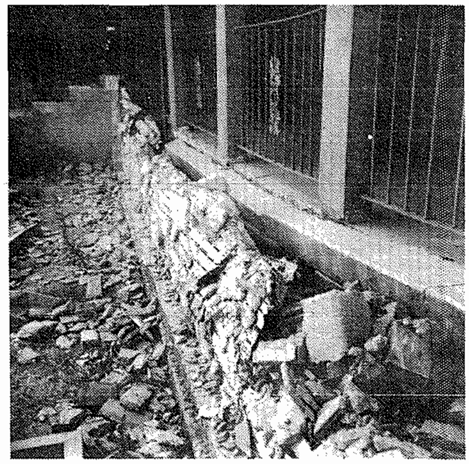

FIGURE 1.8B. The San Fernando aftershocks finished the job of collapsing the "cripple" studs.

The split-level house shown in Figure 1.8A demonstrates another type of failure. Inadequate bracing at the front of the garage again allowed the house to lean and the very common separation at the split level occurred. In this case the house leaned toward the mid-level, indicating that the mid-level had shifted even farther than the two-story portion. The reason for this soon became evident. The mid-level had been constructed using short "cripple" studs to effect the difference in elevation between it and the garage. Veneer had been placed over these studs with only diagonal bracing available between the studs to resist horizontal forces. In subsequent aftershocks these studs completely collapsed, as shown in Figure 1.8B. When this occurred the floor of the mid-level took on the appearance shown in Figure 1.8C. The house was subsequently razed.

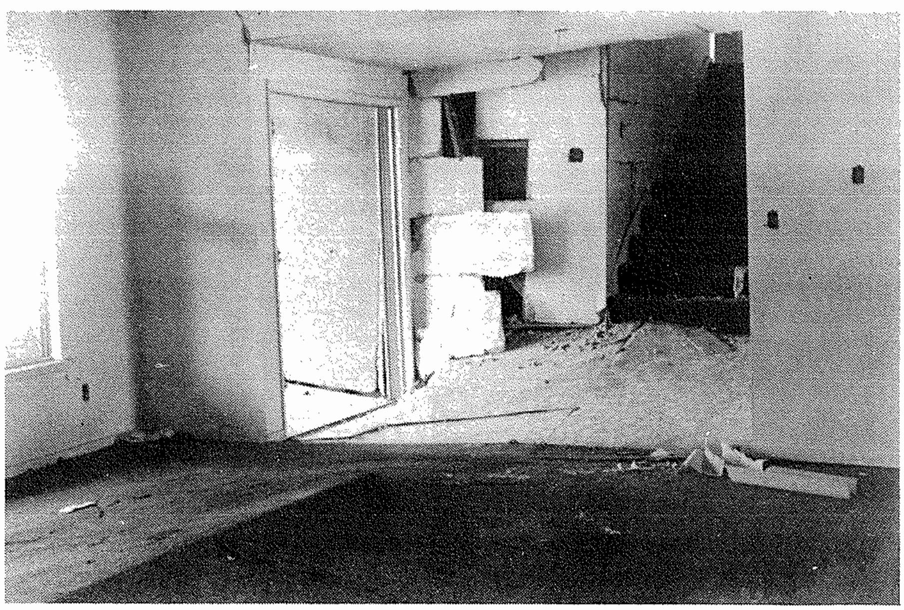

FIGURE 1.8C. The floor of the mid-level was left a shambles.

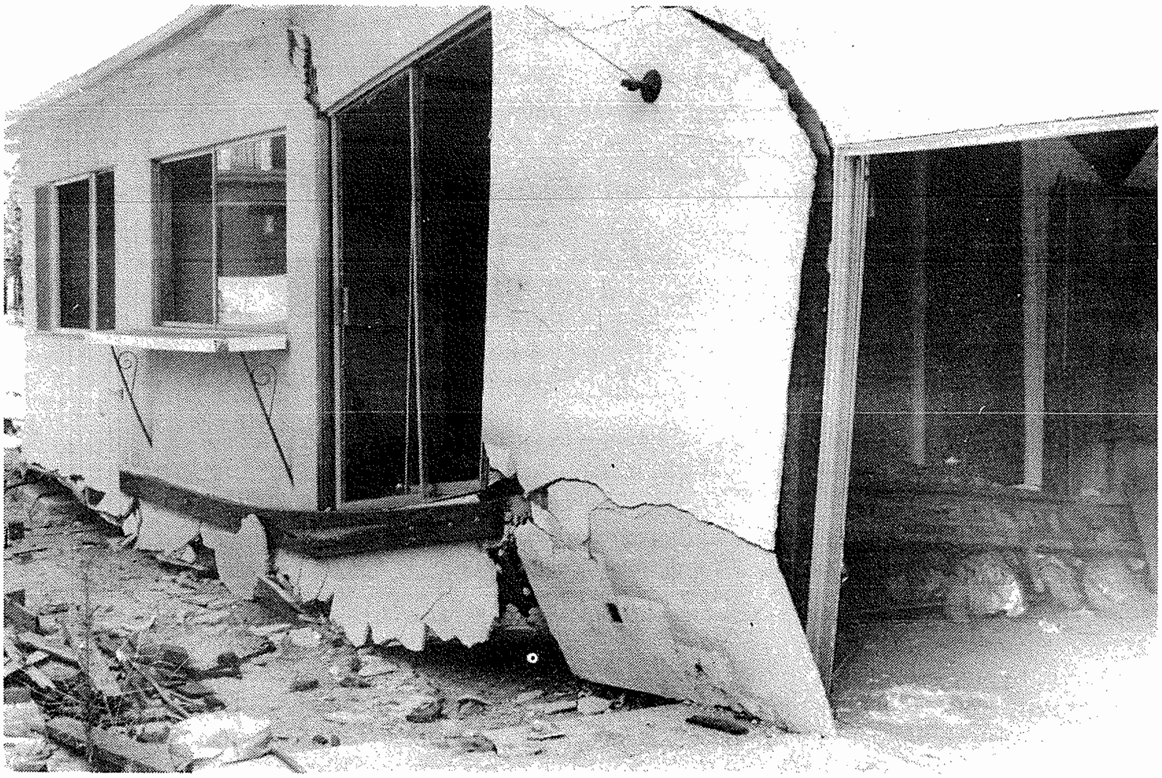

FIGURE 1.9A. Another failure of "cripple" stud walls, with dramatic consequences to the wall (center).

Another example of the failure of cripple stud walls is shown in Figure 1.9A. The failure of the shear wall in the center of the picture is in this instance quite dramatic. When houses of this type fail the reader is given some understanding of the dangers involved as shown in Figure 1.9B. The stairway from the mid-level to the upper level of still another collapsed split-level house is shown in the center of the picture. The family room, now approximately 3'-0" in height, is shown in the lower right hand corner.

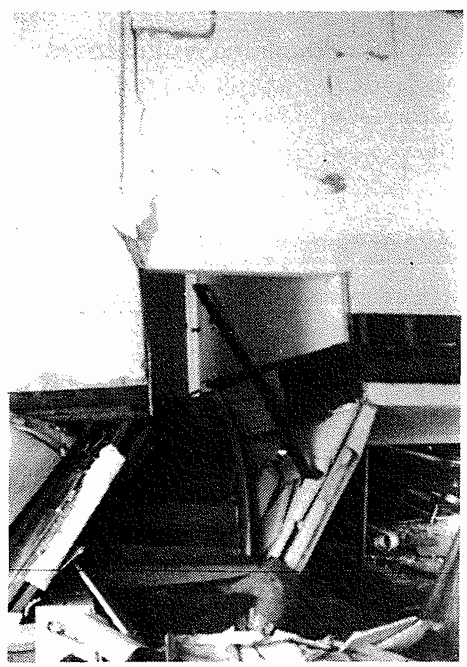

FIGURE 1.9B. A family room in a split-level home with "cripple" stud walls became about three feet high.

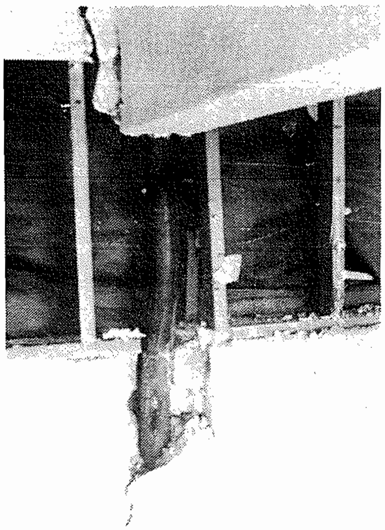

FIGURE 1.10A. (Left)

A "knee wall" withstood the force of a pipe column attempting to move out of the wall.

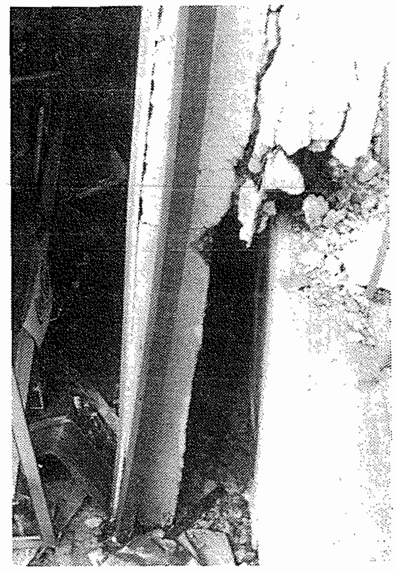

FIGURE 1.10C. (Right)

Furring – not attached to the masonry – in another retaining wall in the building moved several inches during the earthquake.

Known by various names in different parts of the country, the construction consisting of an approximately four-foot high retaining wall with wood studs above will be referred to as a knee wall in this report. Considerable concern has been expressed in some parts of the nation with regard to performance of the retaining wall in construction of this type. Knee walls are not common in Southern California but, on the basis of the structure shown in Figure 1.10A at least, it appears that properly designed walls of this type will withstand earthquake forces. The wall was strong enough to withstand the attempt of the pipe column to move out of the wall and caused considerable bending in the column. The forces did contribute, however, to considerable damage to the stud wall above (Figure 1.10B below).

Another retaining wall in the same building was furred at the inside of an apartment (Figure 1.10C). Furring was not attached to the masonry and moved several inches during the earthquake. This is a very large wood frame apartment complex which, although it did not collapse, received damage assessed at more than one million dollars.

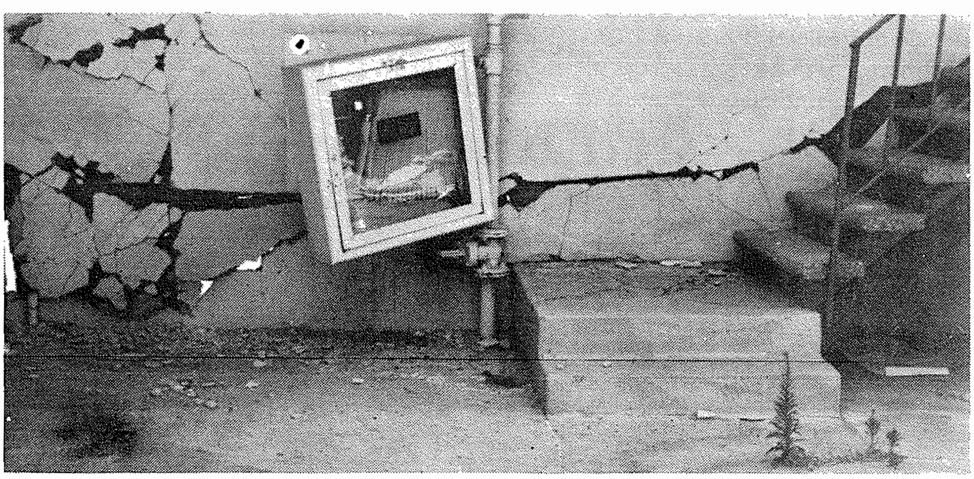

FIGURE 1.10B. Substantial damage occurred at the knee wall but it did not collapse.

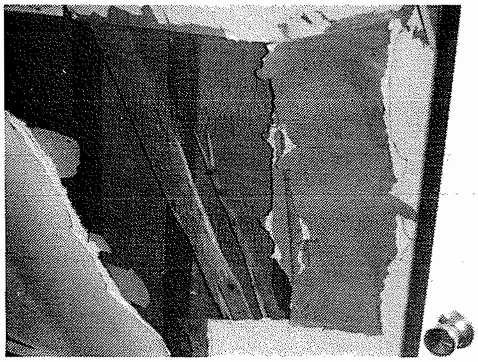

FIGURE 1.11.

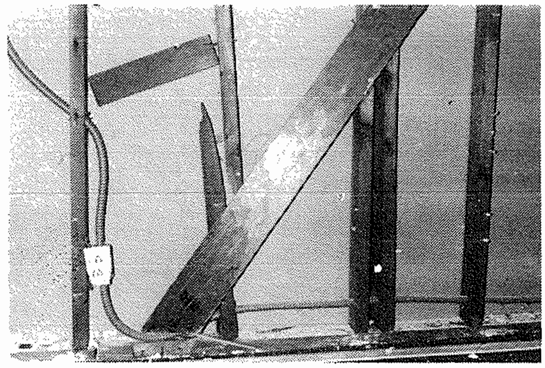

FIGURE 1.12.

Inadequacy of let-in bracing: Tension failure on the left; sill plate pull-out on the right.

Let-in braces were frequently used to assist in resisting horizontal load. These braces proved time after time to be inadequate. They either failed in tension as shown in Figure 1.11 or they pulled out from the sill plate as shown in Figure 1.12.

The San Fernando earthquake provided several examples of nonstructural component damage as well. The strap ties in the fireplace as shown in Figure 1.13 proved to be inadequate and the entire fireplace collapsed in this apartment house. Another total collapse took place in a single-family residence as shown in Figure 1.14. Fireplaces and chimneys failed in a number of ways. Strap ties were required to be connected to both second-floor framing and roof framing, and fireplaces and chimneys were required to be fully reinforced from the foundation to the top of the chimney. In some cases the strap ties were attached to the masonry but not attached to the roof framing. Some of these fireplaces leaned outward from the house as a unit but did not overturn. Others fell completely. In a few cases the strap ties were properly connected but failed. When this occurred the chimney sometimes failed with little damage to the firebox itself; in others the entire assembly again overturned. Where the masonry fireplace occurred at the interior of the house damage was much less severe except for that portion of the chimney extending above the roof. Finally, some fireboxes themselves crumpled, causing total collapse of the assembly.

FIGURE 1.13.

FIGURE 1.14.

Inadequacy of strap ties: Fireplace collapse on the left; chimney and firebox collapse on the right.

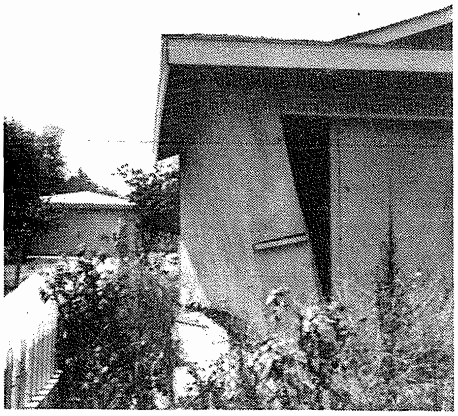

FIGURE 1.15. **Even lightly loaded garages were damaged.**

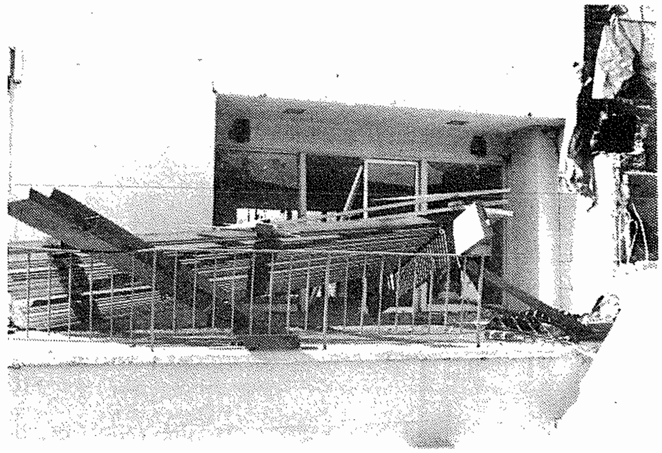

FIGURE 1.16. **A latticed patio roof overhang collapsed because of virtually no lateral bracing.**

Where very little wall existed on either side of the garage door, failure of garages was not dependent on large loads from structure above or other adjacent structures (see Figure 1.15).

Collapse of miscellaneous structures was prevalent in San Fernando. A latticed roof overhang had been constructed over a patio (adjacent to the fireplace shown in Figure 1.13) with virtually no provision for lateral bracing and with the two supporting beams simply connected to the header over the window of the lobby of the apartment house. As might have been expected, the overhang pulled away and collapsed (see Figure 1.16).

Veneer and masonry also created many problems. The house shown in Figure 1.17 had been fully veneered up to the ceiling line, and ties had been installed as can be seen. The danger to anyone standing near the house when the earthquake occurred is apparent and the cost of repair of the damage is also obvious.

Minor structural damage such as the collapse of the porch roof support shown in Figure 1.18 is not a major concern of this report. The value of positive connections at the tops and bottoms of all framing members, however, should be apparent.

FIGURE 1.17. **Veneer damage was perilous, costly to repair.**

FIGURE 1.18. **A porch collapse illustrating the value of proper connection of framing members.**

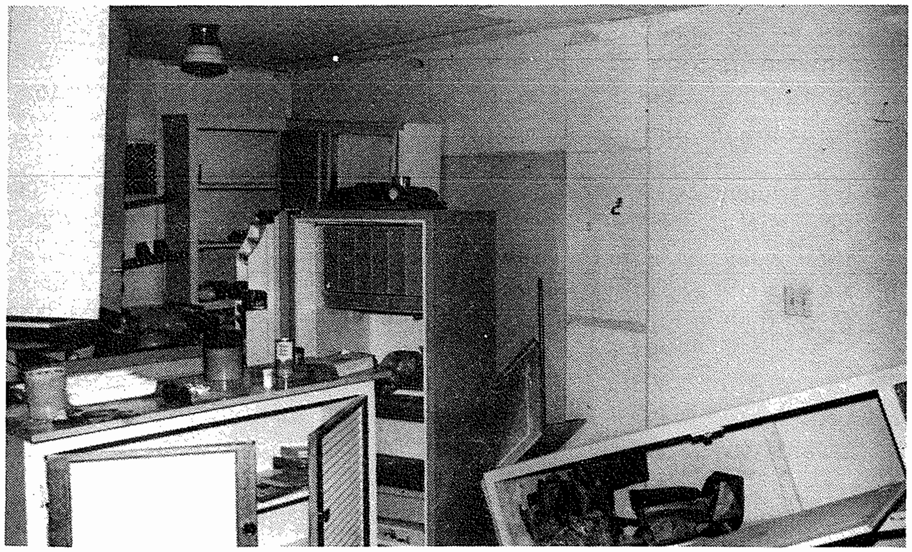

FIGURE 1.19. **San Fernando quake moved heavy refrigerators, toppled cabinets.**

Several types of nonstructural damage are evident in Figure 1.19. The refrigerator has moved a significant distance from the wall. Although it would be desirable if heavy appliances were required to be positively attached, that is beyond the province of this report. Kitchen cabinets are installed as a part of the finished product and the damage shown can be lessened or prevented. Details for that purpose are presented in Part IV of the report.

It would be equally desirable, though not very popular, to outlaw the widespread practice of building bookcases out of concrete blocks; the dangers are apparent in Figure 1.20.

While vibration damage can be somewhat controlled, the vibrations themselves cannot be eliminated from a structure. It is an assumption basic to this report, however, that the more the horizontal movement of a residence is inhibited, the lass damaging and perilous will be an earthquake's vibration forces on its finish materials, its nonstructural features, its esthetic and comfort contents, and on its inhabitants. Perhaps Figure 1.20 points up the need well — to say nothing of the extreme repair costs — with its evidence of severe damage in an area that is certainly of high personal vulnerability.

FIGURE 1.20. **Popular but dangerous bookshelf contrivance.**

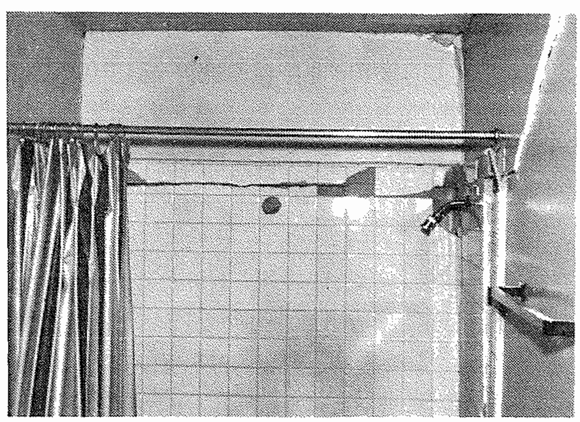

FIGURE 1.21. **Halftones of costly repairs, plus overtones that give one pause.**

Summary of Earthquake-Caused Residential Damage

Following are some of the costly, major weaknesses in design and construction disclosed by the vibration forces generated during the "moderate" San Fernando earthquake – only magnitude 6.6 on the Richter scale:

- Insufficiency of walls with enough rigidity to resist horizontal forces. Many two-story or partial two-story houses suffered severe damage because so large a percentage of their walls contained windows or other openings leaving relatively little to provide rigidity. Provision of walls to supply resistance (shear walls) is one of the key design recommendations developed in this report.

- Improper or insufficient connection of studs to sill plates. This condition, one of the primary weaknesses found in wood frame construction, led to failure of shear walls in several ways:
  - rotation in the plane of the wall (failure in overturning);
  - uplift of the entire wall caused by vertical acceleration and a combination of horizontal forces;
  - outward movement of the base of the wall caused by load perpendicular to the wall.

- Inadequate tying together of the portions of split-level houses.

- Absence of effective wall on either side of garage doors. This proved to be a problem for homes with second floor above the garage as well as for one-story garages.

- Use of short or "cripple" stud walls with only diagonal bracing between studs. Diagonal bracing proved inadequate even for "cripple" walls.

- Numerous miscellaneous faulty design and/or construction features:
  - let-in braces, frequently used to assist in resisting horizontal load, proved time and again to be inadequate, failing in tension or pulling out from the sill plate;
  - fireplaces and chimneys collapsed or were heavily damaged because of a variety of reasons, primarily because of insufficient strap ties or, if they were used, improper or nonexistent connection;
  - improper anchorage of interior and exterior veneer;
  - inadequate anchorage of porch roofs and other overhangs.

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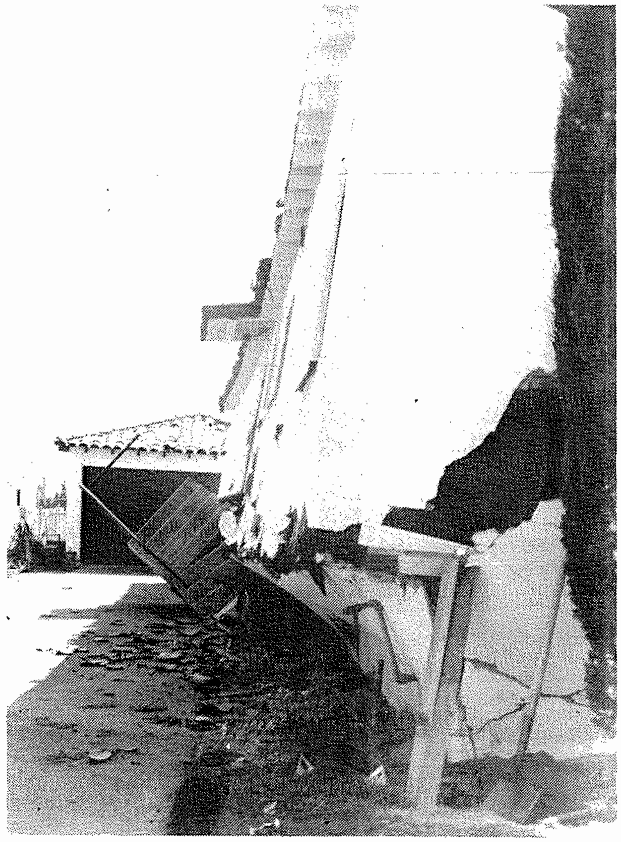
PART II

BASIC SEISMIC ANALYSIS FOR SINGLE-FAMILY RESIDENCES



CHAPTER II–1 PRINCIPLES OF SEISMIC-RESISTIVE DESIGN OF DWELLINGS

To use the recommended design methodology and construction details properly, it is essential that the home designer understand the way buildings act when subjected to earthquakes. This chapter describes how earthquakes impose forces on structures, identifies the components of structures that act to resist these forces, and shows how the components must be constructed and interconnected to function effectively.

In general, terminology used to identify building components is familiar to the designer. To be consistent with phraseology used in common earthquake design practice and to emphasize the earthquake-force-resisting components of residential buildings, some terms are used that may be unfamiliar. The new terms are defined when first used.

It is important to recognize at the outset that principles common to vertical gravity load and wind load design also apply to design for earthquakes. That is, various structural components are required to resist or support certain loads. While gravity load design requires consideration only of loads acting vertically, wind and earthquake design both require consideration of horizontal loads. Though overall design of structures for wind is similar to overall design for earthquakes, wind loads are forces applied to the structure externally, whereas earthquake forces are generated from within the structure.

Earthquake Forces (Inertia Forces)

Earthquakes impart forces to structures because of inertia, the tendency of a body to continue its movement or its lack of movement, in other words to resist acceleration or deceleration. Inertia force, for example, causes the driver of an automobile to be thrust backwards in his seat when the car accelerates rapidly. When the auto is stopped quickly the occupant will continue to move forward because his body attempts to continue at the speed it was traveling.

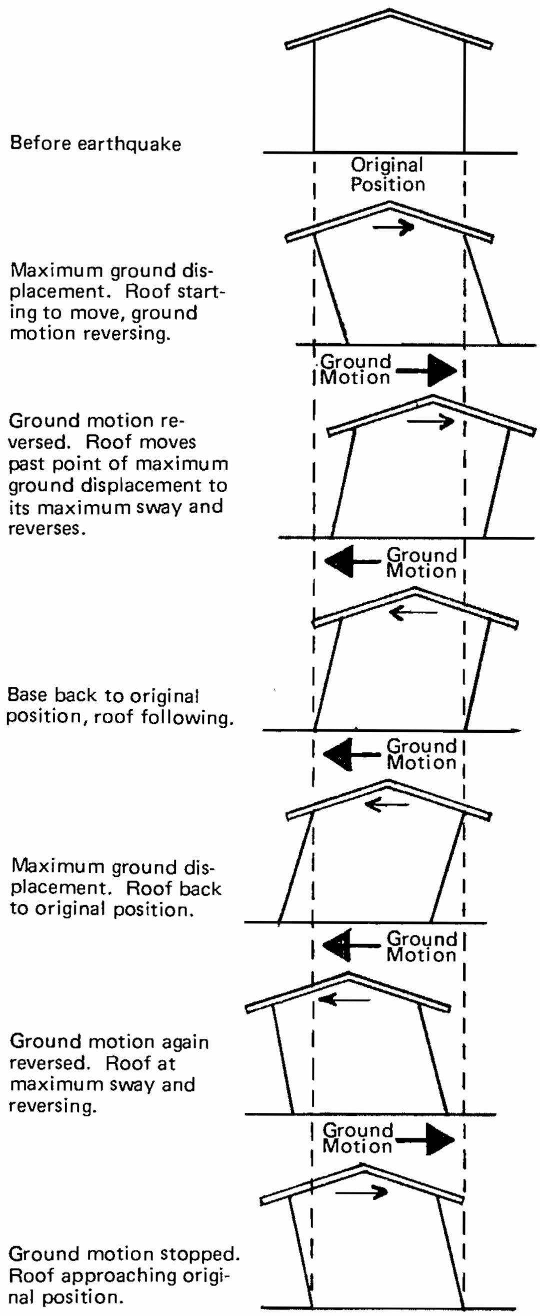


FIGURE 2.1 Responses of a one-story house to earthquake motion — an idealized sequence.

During an earthquake the ground surface in the region affected generally moves back and forth, up and down, in random fashion. This motion can be characterized by how much the ground surface displaces from its original position and by the velocity and acceleration at which the displacement occurs. A structure will be accelerated back and forth by the motion of the ground on which it rests and inertia forces will be generated in the structure similarly to the actions and forces generated in the example of the automobile and driver.

Effects of Earthquake Forces on Structures

When the ground beneath a structure moves, the foundation will move with it because of friction between the foundation and ground. Initially, in varying degree, the upper portions of the structure will tend to remain stationary because of the inertia force acting opposite to the direction of motion and because of the flexibility of the structure. Next, if the strength capacity of the structure has not been exceeded, the "roof" will eventually move in the direction of the foundation movement and, because of the roof's inertial force it will overshoot its mark, passing beyond the point over the foundation at which it started. Then, in an attempt to catch up, the upper portions will be whipped back and forth until the earthquake motion is halted. Figure 2.1 shows a few idealized sequences in the response of a one-story house to horizontal earthquake motion.

Because earthquake waves have vertical as well as horizontal components, the structure moves up and down at the same time it is moving horizontally. When the ground moves upward the foundation is subjected to a slight additional pressure which may be thought of as if the building "weighs" more. As the ground suddenly moves downward the structure may be thought of as "hanging in air" briefly before it follows the ground on its downward path. Since all structures must be designed for vertical gravity load and because of the factors of safety that are normally included, the vertical component of earthquake motion may be ignored. Except in the case of the connection of studs to sill plates, observations of earthquake damage to residential structures have not shown vertical motions to be of significant consequence.

Although it is the ground that moves and the building that tends to follow after, earthquake forces can also be visualized by considering the ground to be stationary with a huge, piston-like force being applied to the building at the roof level forcing it to rock to and fro. The results on the structure are the same whether the force starts at the ground and moves up the structure or starts at the roof and moves down. The piston generates a force that is a function of the machinery driving it; the ground motion causes inertia forces in the building that are proportional to the weight of the structure and the magnitude of the acceleration.

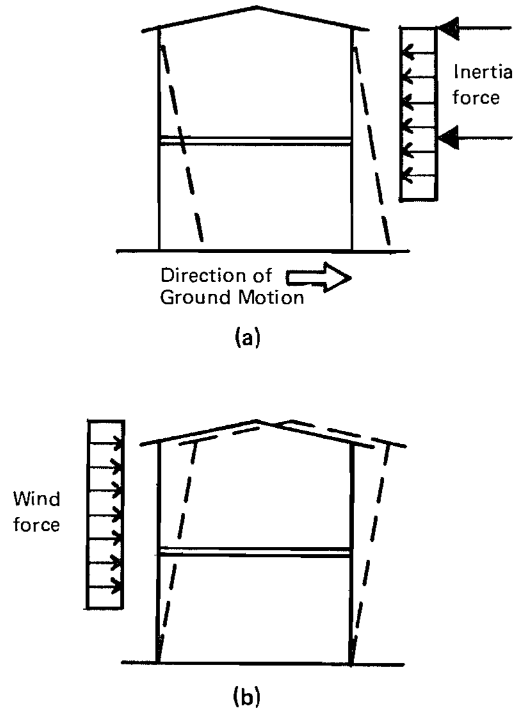


FIGURE 2.2. **Instantaneous distortion of a two-story house due to earthquake and wind.**

In some cases, wind forces are easier to visualize than the inertia forces generated by earthquakes. Figure 2.2 shows similarities and differences between the overall wind and earthquake forces acting on a two-story house. Note that though the wind in Figure 2.2b and the ground movement in Figure 2.2a are acting in the same direction, the houses tend to rack in opposite directións. This occurs because inertia forces generated by earthquakes act in the direction opposite to that of the ground motion.

Inertia forces in a structure are generated by each and every component — walls, floors and roof. The magnitude of the inertia forces is proportional to the weight of each element and the acceleration of that element. For one- and two-story single-family residences, it is reasonable to assume that acceleration is equal throughout the structure. For any given ground acceleration, therefore, variations in the inertia forces throughout a structure may be assumed to arise only from the variations in the weights of the structure components. The inertia forces of the walls of the house in Figure 2.2a are shown as uniformly distributed (small arrows). Because the weight of floor or roof generally is large (and concentrated in the horizontal plane) the forces of these elements in Figure 2.2a are shown as concentrated loads (large arrows). The inertia forces (loads) in the walls, floor and roof must be transmitted through the structure back down to the ground; for either of the structures shown in Figure 2.2, bracing parallel to the direction of load is essential to prevent collapse. The small arrows representing the uniformly distributed inertia force and wind force are not shown as extending down to the foundation in Figure 2.2 because the forces at the bottom half of the first story wall are usually assumed to be transmitted directly to the foundation and therefore do not enter into design of force-resisting walls.

Lateral Force Resistance Systems for Dwellings

The principal bracing normally provided in houses to resist earthquake (or wind) loadings is a system of shear walls. Shear walls are walls parallel to the direction of earthquake motion that act as vertical beams to carry the lateral (inertia) loads to the ground.

For shear walls to do their jobs, loads developed elsewhere in the structure must be carried to them. Since the seismic loads are either developed in the floor and roof systems themselves or are carried to them by other components of the structure, each floor and roof must act as a unit, as if it were a horizontal beam, to carry the loads to the shear walls. (A floor or roof acting in this manner is commonly called a diaphragm.)

Walls perpendicular to the direction of ground motion are subject, of course, to inertia forces (Figure 2.3) and must be supported by other parts of the structure. Each stud of the perpendicular wall acts as a beam and transfers its load to the top and bottom plates. The resistance to the inertia force is provided through the connection between the top plates and roof and between the bottom plate and the floor as shown in Figure 2.3. Thus, one half of the load generated by the perpendicular walls is resisted by the roof above and the other half is resisted by the floor below. The resistance provided by the connections between the plates and floor or roof obviously is dependent on the nailing.

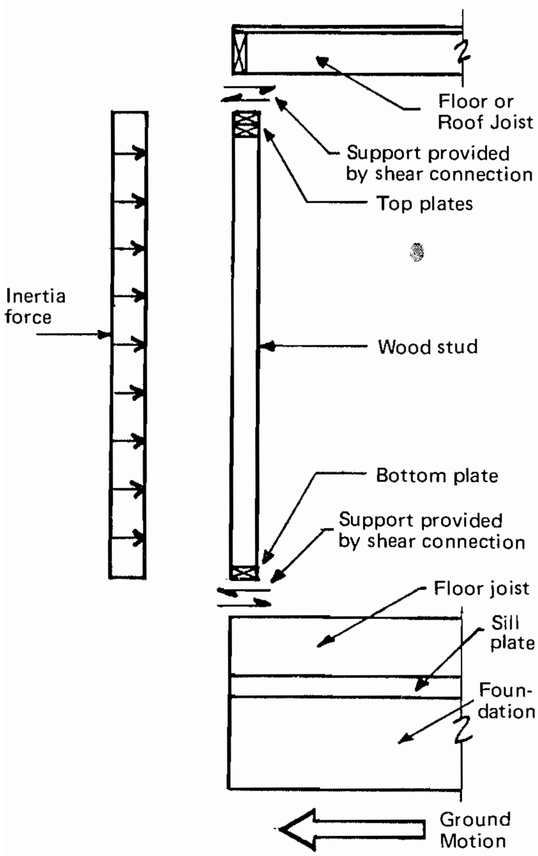


FIGURE 2.3 **Inertia forces acting and resistance for walls perpendicular to direction of ground motion.**

Typically, a diaphragm (roof or floor) will be called upon to resist the inertia force generated within it as well as portions of the load from walls perpendicular to the direction of motion. The schematic of a one-story flat-roofed house in Figure 2.4 illustrates how inertia forces from the perpendicular walls and the inertia force generated within the diaphragm itself are resisted by the shear walls through the connection between the diaphragm and the shear walls.

A shear wall, in turn, must resist not only the inertia force transferred to it from the diaphragm above but also the inertia forces generated within itself. Figure 2.5 shows how these forces act on a shear wall and how resistance to the forces is provided by the connection between the shear wall

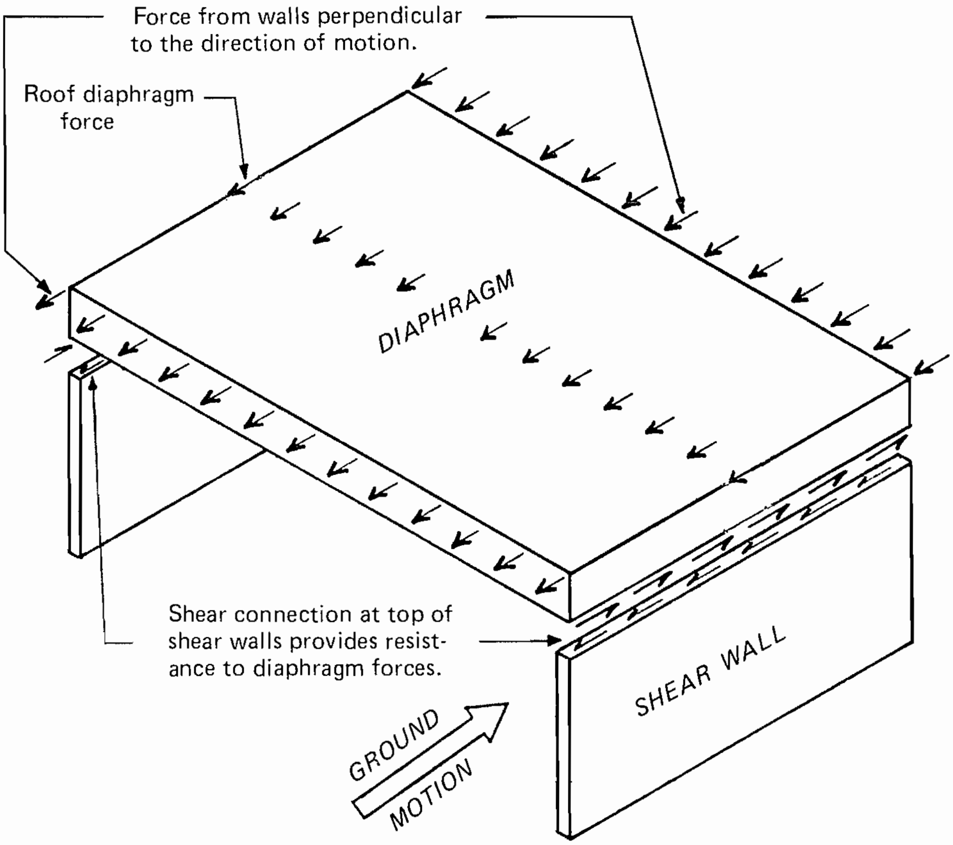


FIGURE 2.4. **Inertia forces acting and resistance of roof diaphragm.**

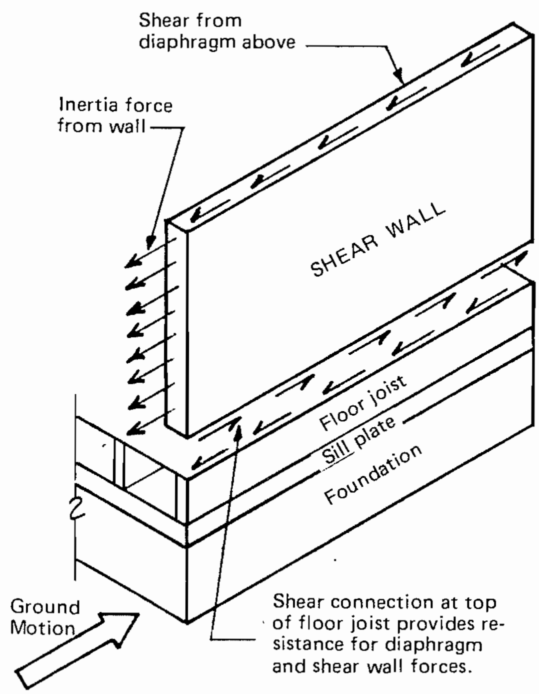


FIGURE 2.5. **Inertia forces acting and resistance of walls parallel to the direction of ground motion.**

and the floor. Although not explicitly shown in Figure 2.5, the transfer of the shear through the floor to the sill plate, to the foundation and, finally, to the ground must also be provided for.

The discussion thus far has introduced three principal components of a house that must be properly designed for the house to be earthquake resistant: Diaphragms, Shear Walls and Connections. If any of these components functions inadequately, the structure likely will suffer excessive damage or even collapse during an earthquake. Following is a more detailed discussion of each of the components in terms of function and construction requirements.

Diaphragms

As previously stated, when a floor or

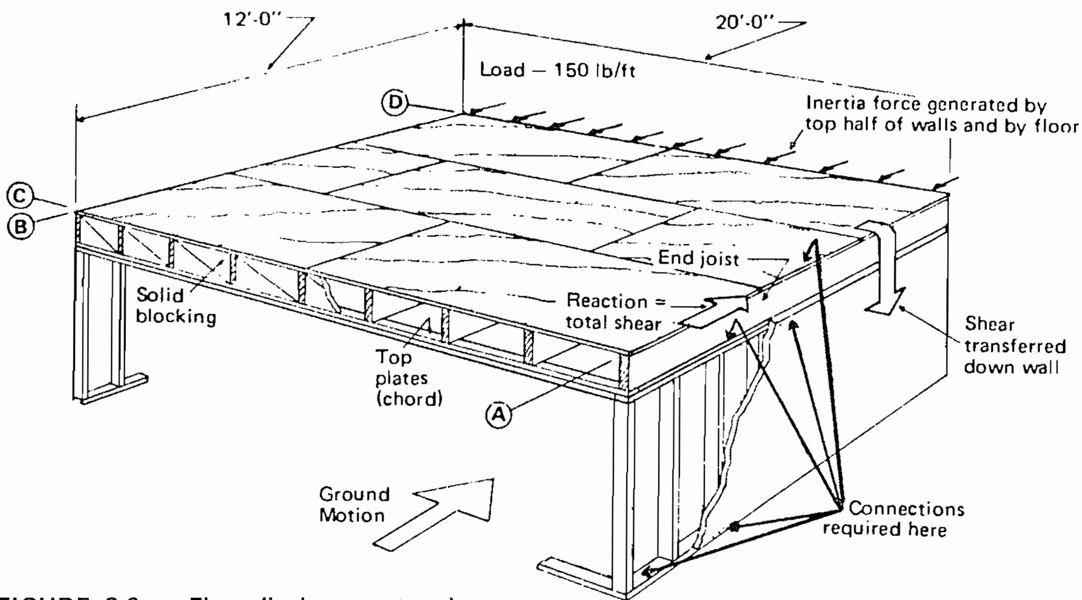


FIGURE 2.6. Floor diaphragm acts as beam.

roof system is required to act as a beam in the horizontal plane to carry lateral forces it is termed a diaphragm. Using a building with no interior walls as an example (see Figure 2.6), the diaphragm shown consists of the floor sheathing, the floor joists, the blocking between the floor joists at the joist ends and the top plates on which the floor joists rest. For clarity, walls perpendicular to the direction of motion are omitted. The lateral forces the diaphragm must support consist of the inertia force from the walls perpendicular to the direction of ground motion and the inertia force generated by the diaphragm itself. The two end walls (Walls A and B in Figure 2.6) are the shear walls that transfer the lateral force from the diaphragm to the ground.

As do other beams, diaphragms resist or carry load through the combined action of shear and bending. Although shear forces may be quite complicated in a full engineering analysis, the complex aspects are usually obviated in the design and construction of single-family residential dwellings by such elements as the plywood sheathing and its nailing plus other required connections. For the purposes of this discussion, shear may be thought of in its simplest aspect: The stress developed at a section or plane where one part of the body attempts to slide with respect to the adjacent part. Consider, for example, the failure of a light pole struck by an auto because the material of the pole could not transfer the force to its base where it might be resisted and, instead, is "sliced" off. If the light pole were struck with less force it might remain intact because it could transfer the force of the lateral impact down its length to where it is anchored. Carrying the example further, it is possible that the pole could successfully transfer the load but the total structure could fail if its connection, the anchoring bolts, fail in shear.

The manner in which forces are resisted or transferred through roof and floor diaphragms (horizontal diaphragms) or walls (vertical diaphragms) may also be illustrated by discussion of the action of common, familiar beams:

Consider first a comparison of wood beams each supported at both ends and each with a concentrated vertical load applied at the center of the span (see Figure 2.7). The beam in the upper diagram, Figure 2.7(A), consists of four planks laid one atop another and the planks bend under the load somewhat independently as indicated by the offset at the ends of the planks. Since the planks are acting as a parallel system, it requires little imagination to realize that the four planks will be four times as strong as one plank. In the lower illustration, Figure 2.7(B), the four planks have been laminated (or bolted) together. The laminated beam is much stronger than the four individual planks as indicated by the lesser deflection of the lower beam. The lower beam is much stronger because, in laminating the planks together slippage (or shear deformation) between the planks is prevented and they bend as a single unit. Theoretically, it can be shown that the laminated beam is four times stronger than the stacked planks. Bending of the laminated beam as a single unit is demonstrated by the straight lines at the ends of the lower beam. Note again that the ends of the individual planks in Figure 2.7(A) are offset – indicating that each plank acts independently in bending. Inspection of the laminated beam also reveals that the line representing the bottom of the bottom plank has lengthened. The initial length of the beam before bending is represented by the center line of the five lines shown. Because the upper two lines are now shorter and the lower lines longer, it follows that the upper half of the beam is in compression and the lower half is in tension. Relating this to the upper diagram containing the four unlaminated planks, it can further be seen that a center line drawn through any of the four planks represents the initial length of the plank before bending, and that each plank is likewise in compression at its top surface and in tension at its bottom surface. This explains the offset at the ends of the planks shown in the figure.

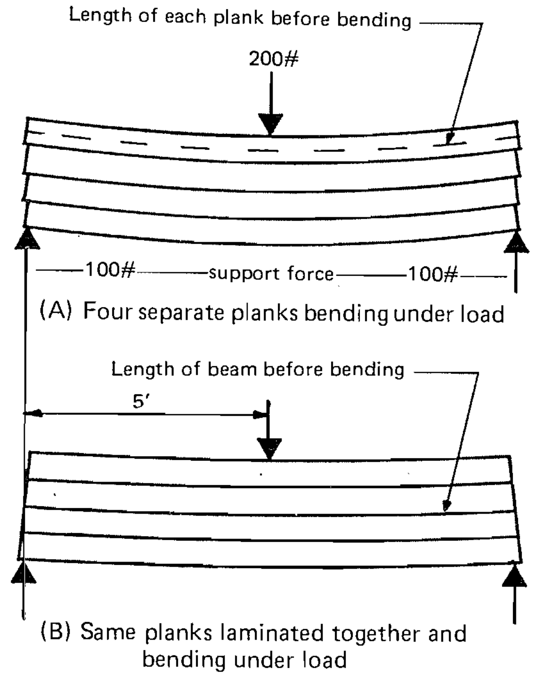


FIGURE 2.7. **Shear and bending in a laminated wood beam.**

Consider also a standard steel wide-flange beam supported at both ends and carrying a load distributed uniformly along its length (see Figure 2.8). The development of this type of beam is based on the engineering principle that the flanges take the tension and compression arising from bending as described for the plank-beam examples, and

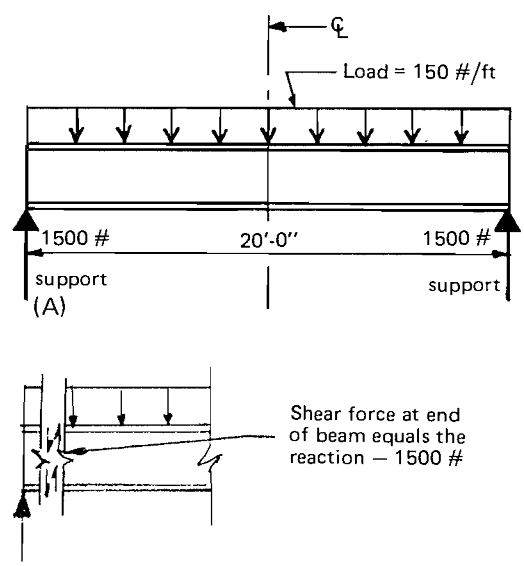


FIGURE 2.8. **Shear in a steel beam.**

the web carries the shear developed between the flanges. For a diaphragm to function properly it must be competently assembled and must include the shear-carrying capability of the web and the tension- and compression-carrying capability as in this wide-flange beam example. Since the load applied to the beam in Figure 2.8 is uniformly distributed, the support force (or reaction) required at each end is equal to one half the total applied load or 1500 lbs as shown in the example. The load to the left of the center line of the beam is carried to the left-hand support and the load to the right to the right-hand support. The load per foot on each side of the center line is transferred through the beam to the support through shear and bending stresses within the beam. Shear and bending stresses in beams vary throughout the length of a beam. The maximum shear in a simply-supported beam occurs at the ends of the beam and would be equal in magnitude to the support reaction (1500 lbs) as shown in Figure 2.8(A). Bending in beams can be similarly quantified but bending stresses in diaphragms are low for most residential construction and need not be quantified, and therefore will not be discussed.

Bending in diaphragms is resisted primarily by the diaphragm's chords which serve the same function as the flanges in a beam. This is demonstrated by referring back to Figure 2.6 in which the top plates function as the diaphragm chords. With the direction of ground motion indicated in Figure 2.6, the chord along Line C is in tension, the chord along Line D is in compression. If the ground motion were changed, of course, the reactions would be reversed and because earthquake ground motions oscillate back and forth, chords must be designed to resist both tension and compression stresses. Because tension and compression stresses in the chords of residences are relatively low there need not be great concern with the design of chords, but it is essential that the chords be continuous members for the entire width of the diaphragm. The required continuity is achieved by use of two top plates, with offset splices where appropriate, and sufficient nailing. Because the chords are not connected directly to the sheathing it is the function of the blocking between the joists to transfer the tension or compression, through nailing, from the sheathing to the plates.

Shearing stresses in diaphragms are carried by the sheathing (plywood or other material) which functions as the web of a beam. Because the sheathing for an entire diaphragm is not a single piece, as in the steel beam example, shear must be transferred from one plywood sheet to the next by adequate nailing to a common member—in this case, joists. It is the nailing that tranfers the shear from the plywood into the member connected to, or a part of, the shear wall.

Two other concepts involved in the nature of shear forces in diaphragms and how they are resisted are "shear-per-foot" and "diaphragm ratio."

Shear-per-foot is maximum shear force divided by diaphragm depth. Because the maximum shear force in a diaphragm occurs at the diaphragm's supports, shear at intermediate points within the diaphragm need be of no concern. Again using the diaphragm shown in Figure 2.6 as an example, with the hypothetical load indicated, the total applied lateral force is 150 lb/ft x 20 = 3000 lbs. The support reaction at each of the two shear walls is, therefore, 3000/2 = 1500 lbs. For this example the maximum shear-per-foot in the diaphragm is 1500 lbs / 12′ = 125 lb/ft.

Diaphragm ratio is the ratio of the width of the diaphragm (distance between shear walls) to the depth of the diaphragm. For the diaphragm in Figure 2.6 the diaphragm ratio is 20:12 (or 20/12) = 1.67.

Allowable shears per foot for plywood and other diaphragm materials are tabulated in building codes and elsewhere for various thicknesses and grades of materials for nail spacings from 2" to 6" o.c. and for certain limitations on diaphragm ratios. Although the nailing of floor and roof diaphragms is important, the shear stresses in single-family residences are usually low enough to allow the use of the maximum nailing space given in the codes. Requirements for nailing and details for placement of diaphragm sheathing are presented in Parts III and IV of this report.

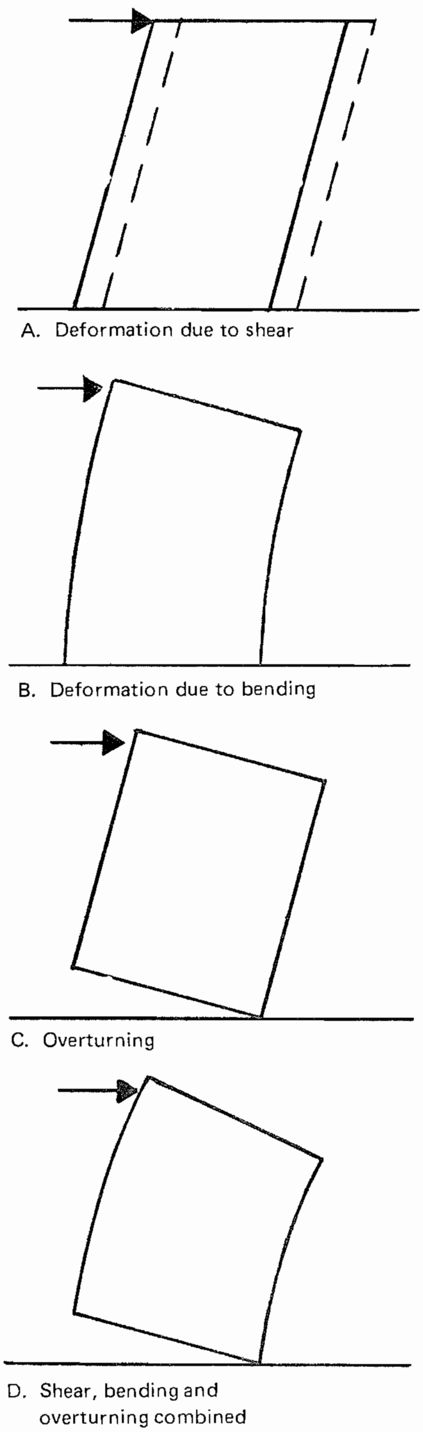


FIGURE 2.9. Action of a shear wall under horizontal load.

Shear Walls

The second principal component of a house requiring proper design and location for earthquake resistance is the system of shear walls – the walls that transfer to the sole plates the inertia forces applied to the walls by other building components, principally diaphragms, plus the forces generated within the walls themselves.

The vertical plane of a shear wall is termed a Line of Shear Resistance. A line of resistance may consist of one shear wall or more than one if the walls effectively act together in transferring shear. At least two parallel lines of resistance – one near each end – are required to provide stable support for the diaphragm. And since, as has already been noted, earthquake ground motion may occur erratically in all directions, lines of resistance must be provided at or near all four edges of a diaphragm.

Shear walls act as cantilever beams in resisting lateral forces. A cantilever beam is distinguished from a simple beam in that the cantilever if "rigidly" supported at one end only, while the simple beam, as in the case of a diaphragm, is simply supported at both ends.

A shear wall, in effect a vertical diaphragm, acts as a beam and as in the case of the diaphragm must be designed to resist load through the combined action of bending and shear. In a wood stud shear wall the finish material on one or both sides of the wall acts as the web of the beam. The continuous studs nearest each end of the wall act as the flanges.

Various reactions of a shear wall to a lateral force applied at its top are illustrated in Figure 2.9.

- The wall may undergo shear deformation (solid lines of Figure 2.9A) if it is a wall having low shear resistance as would be true, for example, if the finish material were removed, leaving only the studs and the top and bottom plates. Although an uncovered wall is a highly extreme example, it is important to note that, because the shear-resisting materials used in wood frame construction are relatively weak and because the nails or other fasteners tend to slip slightly, this type of deformation is the most prevalent kind of deformation of securely anchored walls caused by earthquake loads.

If a wall is not properly anchored to the floor below, it will tend to slide. The dashed line part of Figure 2.9A illustrates the combined result of wall shear deformation and sliding. Shear transfer must be effected between the bottom of the wall and the floor to prevent sliding.

- A wall may deflect in bending, a phenomenon that can be visualized if the wall is considered to be acting as a beam that is firmly anchored to the floor (see Figure 2.9B). Although shear walls in homes deform in bending to some extent, bending deflections are relatively minor and most design guides base the strength of a wall on its shear-resistance capacity.

- A wall may overturn as shown in Figure 2.9C if it is not firmly anchored to the floor. Whether the wall will fail in overturning because of a specified horizontal force is a measure of the wall's stability. Resistance to overturning is accomplished through the connection of the wall to the floor and/or foundation and is discussed in more detail later.

- A wall may fail in the combined action of shear, bending, sliding and overturning as shown in Figure 2.9D.

Designed shear walls minimize these types of deformations and therefore substantially restrict damage due to vibration — not only in the walls themselves but in most other portions of the structure as well.

Resistance to Shear. Finish material attached to the framing provides a wall's principal resistance to shear. Generally, the capacity of a wall to resist lateral forces is prescribed by its rated shear-per-foot, which is determined by dividing the seismic load acting on the wall by the wall's length. Rated allowable shear-per-foot has been established for a number of finish materials. For example, 5/16" Structural I plywood sheathing attached at all panel edges with 6d nails spaced at 6" has an allowable shear rating of 200 pounds per foot of wall length. Thicker sheathing and/or closer nail spacing will provide higher allowable shear values.

Referring to Figure 2.6, the total lateral load that must be resisted by the shear walls (neglecting the force generated by the shear walls themselves) is 150 lb/ft x 20' = 3000lb. The lateral load that each shear wall must resist is, therefore, 3000lb ÷ 2 = 1500lb. The shear-per-foot in each shear wall is 1500 lbs / 12' = 125 lb/ft. The shear-per-foot rating for the material cited above is in excess of this and, therefore, would be adequate.

Figure 2.10 shows a diaphragm and shear wall system similar to that shown in Figure 2.6. It is larger than the system discussed earlier and the shear wall instead of being "solid" has openings for windows and door. The illustration can represent the first-story framing of a two-story house, with the diaphragm shown being the second floor. Although both walls along Lines C and D are shear walls provided to resist the ground motion coming from the direction shown, consider only Line C for discussion. The solid wall portions of Wall C consist of three 4-ft-long panels and one 2-ft-long panel. (Each of the 4' segments would be

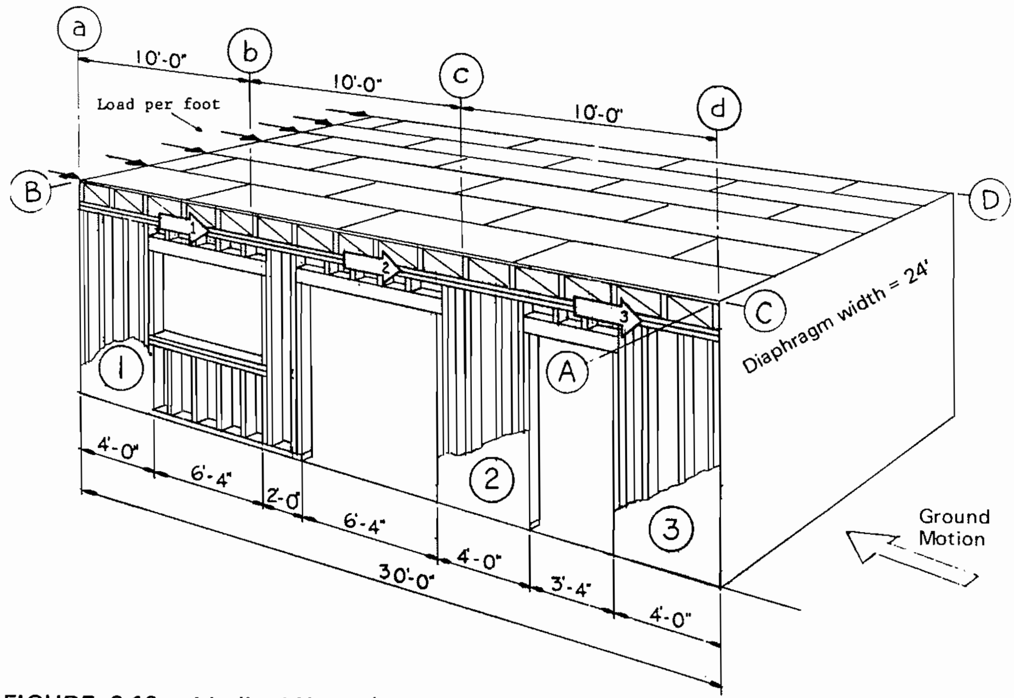


FIGURE 2.10. **Idealized House (Shear Walls and Diaphragm).**

considered to be a shear wall; the 2' panel is too limber in bending to resist its proportional share of the force and would not be considered to be acting.) Wall segments 1, 2 and 3, therefore, must resist all the lateral load imparted to Line C. Since each segment is the same length (4'), each will resist one-third of the load. If total shear force along Line C is 4500lb, the shear that each segment will have to carry is 4500/3 = 1500lb. The shear-per-foot in each segment (wall) is 1500lb/4' = 375 lb/ft. (A quicker way of arriving at the 375 is simply to divide the total shear force in the diaphragm by the total length of shear wall available, 4500/12.) The allowable shear-per-foot rating for the 5/16" plywood, for example, is less than 375 lb/ft and a more resistant material would be required. Shear deformation for walls is almost exclusively dependent on the type of material used and the shear-per-foot acting. For this reason, the lengths of two or more walls along a line of resistance, irrespective of whether they are of equal length, may be added together in determining the shear-per-foot from a given force applied to that line.

It should be noted that in the example in Figure 2.10 the ground motion is perpendicular to the second-floor joists and the chords are the top plates of the end walls, A and B. For the direction of the motion shown, the chord in the near wall, Wall A, is in tension and the chord in the far wall, Wall B, is in compression and the joists at either end transfer tension or compression from the diaphragm sheathing to the chords. The blocking transfers the shear from the diaphragm to the top plates and thence to the shear walls.

Resistance to Sliding. The capacity of a shear wall to resist sliding depends on the connection of the wall to the floor or foundation. The force to be resisted at the bottom of the wall consists of the total shear in the wall including the inertia force transferred to the shear wall from the diaphragm and the inertia force generated within the shear wall itself. This total force must be transferred from the bottom of the wall ultimately to the foundation through a shear flow path indicated in the discussion of Figure 2.5. The shear transfer must be developed by proper nailing of the bottom plates to the floor (by anchor bolts into the footing for slab-on-grade floors) or by extending the shear-resisting material past the sole plate directly to the mudsill which, in turn, must be bolted to the footing.

Resistance to Overturning. The capacity of a wall to resist overturning also depends on its attachment to the foundation but in a manner quite different than the manner used for shear transfer.

Because most of the inertia force generated by an earthquake is imparted to a shear wall near its top, the wall has a tendency to overturn (as in Figure 2.9C). Figure 2.11A shows a hypothetical shear wall with a lateral force P applied at its top and a uniformly-distributed gravity load action downward of w pounds-per-foot comprised of the weight of the wall plus any vertical gravity load such as the roof framing. The force P required for overturning is a function of the total vertical load and if the wall does not overturn it is said to be stable for that specific horizontal force. A larger force applied to the same wall clearly could cause it to become unstable.

Stability of a shear wall is based upon the following facts:

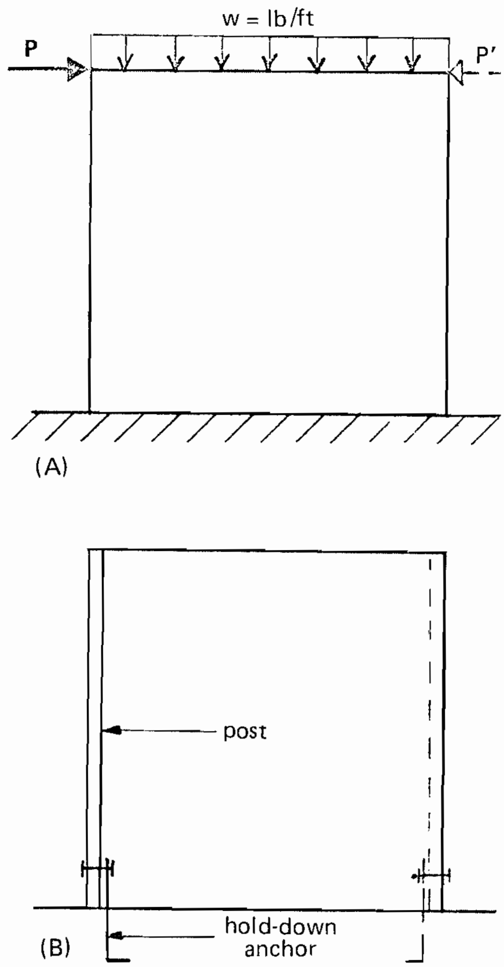


FIGURE 2.11. Wall overturning resisted by hold-down anchors.

- The amount of vertical load on a shear wall (usually expressed as load-per-foot) affects its stability when it is subjected to a given horizontal force.
- Since horizontal design loads for a given wall become a fixed number in seismic design, whether or not a wall is stable depends on the size of that horizontal force, the distance from the base of the wall up to the force, and the vertical load on the wall.
- Since vertical load to walls is usually on a per-foot basis, more vertical load is applied to longer walls than to shorter walls. Longer walls are therefore more stable than shorter walls when a given vertical load is applied to both.
- Stability is also a function of the distance from one edge of the wall to the center of gravity of the total vertical load. This distance is assumed in this report to be one-half the length of the wall.

Since total vertical load and distance to the center of gravity are each dependent upon length, overturning resistance increases faster than length – for a given vertical load-per-foot, a wall 8' long, for example, has four times the overturning resistance of a wall 4' long.

Resistance to overturning tension, when required, should be developed by attaching the chords (end studs) to the foundation. When uplift is transferred to the footing a sufficient amount of the footing must become involved such that the footing's weight is sufficient to resist the uplift. Using an abbreviated description, the end result is that bending is transferred from the wall to the footing and the footing must be reinforced for this bending. A continuous footing involved in the resistance to overturning is termed a grade beam herein.

When it is determined that a wood stud shear wall is unstable, resistance to uplift usually is accomplished with hold-down anchors, straps or framing anchors, as shown in Figure 2.11B. These anchors have been used rarely in residential construction and normally are unnecessary for the types of load generated in one-story residential structures with reasonably long shear walls. When the wall is high and not very long, as is sometimes found in houses with cathedral ceilings, or when the load is heavy due to the weight of two-story construction with relatively little shear wall along a given line of resistance, however, hold-down anchors are frequently required. (Installation of hold-down anchors is discussed in other chapters; because their installation can be bothersome and expensive, particularly if installed in the wrong location, the designer will be shown how to avoid their use for many conditions.)

Locating Shear Walls. Just as important as the shear walls themselves is their proper location, their position relative to the physical boundaries of diaphragms and to the plan configurations of houses.

Inasmuch as they already occur in most currently designed residential structures it is not particularly penalizing to the average house to require that shear walls be provided along all exterior lines. The fact previously noted that earthquake ground motions strike in all directions points directly to the necessity that diaphragms be supported by shear walls at or near all four respectively perpendicular edges. Since wings and other such irregularities in plan occur, it is also necessary that these be supported for horizontal motion. In some cases wings, etc., can be considered as cantilevers off the main diaphragm. In most such instances, shear walls must also be provided at protruding portions of the floor plan as well.

Because inertia forces generated in a structure are proportional to weight, it is desirable that the shear walls be located to provide maximum resistance where the forces are maximum. If they are not, the diaphragm will tend to rotate, causing a torsional effect that will rack the wall at one end of the diaphragm more than that at the other. Except for the small additional loads to a particular line of resistance from wings or masonry fireplaces, loads in residences are quite uniform. Even where irregularities in diaphragm sizes occur any increase in diaphragm length is usually accompanied automatically by an increase in overall exterior wall length. Because some rotation will occur in almost any diaphragm, it is recommended that walls be provided at or near the corners of the structure. Walls in these locations – especially if they are long enough to qualify as shear walls – help to resist rotational effects.

It is recommended that rather than concern himself over where loads are apt to be heaviest, the designer should strive for "balancing" shear walls, even in walls with the same overall length. If, for example, the two-story portion of a split-level home is 21'-0" wide, constantly, from front to back, it is quite conceivable that the shear wall at the front might be a 4'-0" wall adjacent to the garage door, while the wall at the rear might be "solid" and

21′-0″ long. The load to each wall would probably be equal and, since shear deformation is a function of shear-per-foot, the deflection of the 4′-wall could be more than five times as great as that of the 21′-wall. Such an imbalance would cause rotation of the diaphragm and could result in severe damage. It would not be facetious to say that it would actually be better to design an opening in the rear wall to reduce the amount of shear wall available and more nearly balance the walls. It is recognized that the demands for style and amenities in the modern home often make it impossible to locate and size shear walls to achieve anything approaching absolute symmetry but the imbalance described in this example should be avoided wherever possible.

Struts

Struts are sometimes necessary to transfer load from the horizontal diaphragm to a shear wall or line of shear resistance. Although ordinarily the designer need not concern himself with the allowable shear-per-foot in horizontal diaphragms, such limitations do exist. For instance, the allowable shear for 1/2″ Structural II plywood nailed with 10d nails at 6″ o.c. at all edges of sheets is 290 lb/ft. It is conceivable that the shear wall along one or more edges of a diaphragm might be so short that the shear-per-foot in the wall exceeds this value. If shear were only transferred from the diaphragm directly above the wall into the wall, the allowable shear in the diaphragm could be exceeded. Moreover, when another element is available all along the edge of the diaphragm (such as the continuous top plates) the tendency of the diaphragm is to transfer its load to this element on a per-foot basis.

Figure 2.10 demonstrates how shear walls can be made effective by means of the "collector" action of a strut. As was noted earlier, the total shear is 4500 lbs. Because of the strut action of the top plates the shear-per-foot in the diaphragm is this load (4500) divided by the length of the diaphragm (30′) or 150 lb/ft. If the continuous top-plate strut were not present the maximum shear-per-foot in the diaphragm would be 4500 lbs divided by 12′ (the total length of the three shear walls) equalling 375 lb/ft.

Because of the strut or "collector" action of the top plates the diaphragm load is transferred to the shear wall on a per-foot basis. The top plate picks up the load from a to b for Wall 1 and, considering the direction of load indicated, the top plate is in tension from the right-hand side of the wall over the window to Line b, in other words, the top plate is pulling the wall in the direction of load. The total diaphragm load developed between points b and c contributes the total load in Wall 2. In this case, the top plates are in compression pushing against Wall 2. It should be noted that point c is above Wall 2. Despite this, all the load developed between points c and d contribute load to Wall 3, with the top plate again being in compression. In all three cases the top plates act to collect the load from the diaphragm and transmit it to the walls. All walls so interconnected deflect the same amount and therefore "distribute" the load. This, then, is a second illustration of the importance of proper splicing of top plates.

In the example used, the strut would be automatically installed during construction. In some cases, particularly when interior walls are used as shear walls, it is necessary to provide a strut that is not otherwise built into the system. This is particularly true when a shear wall is not located under, but merely abuts, a diaphragm for which it is providing resistance or when only a small portion of the wall falls under the diaphragm such that allowable diaphragm shear would be exceeded if no strut were provided. Design recommendations in this report attempt to avoid the use of struts for any conditions other than those shown in Figure 2.10.

Connections

The third, and equally indispensable, aspect of seismic-resistive design and construction of dwellings insists that the various elements of the structure be properly tied together. No mechanical contrivance, whether it be an automobile engine or a lateral load-resisting system, will function properly without all its interrelated parts, nor will it function well or for long unless the parts are adequately interconnected.

The critical role played by the connections in house design and construction can well be described by a summary review of the way a lateral load-resisting system functions.

- Earthquakes generate inertia forces within the structure proportional to weight and acceleration.

- Since walls span to either the floor or roof levels, for design purposes all load may be considered concentrated at these elevations. Each floor and roof must act as a horizontal beam to carry the load to walls parallel with the load. When a floor or roof system acts as a horizontal beam, it is called a diaphragm and must be designed to function in much the same manner as any other beam. Diaphragm sheathing transfers the shear and is stiffened against buckling by floor joists or roof rafters. To transfer shear from one piece to the next, adjoining pieces of sheathing must be nailed to the same joist or rafter. Flanges of the "beam" are provided by the top plates (in wood frame structures) which act as chords. The blocking or end joists transfer the tension or compression from the sheathing to the chords.

- Walls parallel to the direction of load act to support the diaphragm and are termed shear walls. Blocking or end joists directly above the shear wall must be properly connected to the sheathing and the top plates to transfer the shear from the diaphragm to the wall. When walls are interrupted, the top plates also act as struts to bring the overall wall into play. Shear walls must be designed so that an adequate amount of shear-resisting material is applied to transfer the load from the diaphragm above to the floor below. Finish materials must be properly connected to the studs and plates.

- To resist sliding, proper shear transfer must be provided from the base of the wall to the floor or foundation, through nailing or bolting or both.

- Walls must be designed with proper connections to the floor or foundation to resist overturning.

Essential keys to proper design of lateral load-resisting systems are the determination of adequate shear-resisting materials and the installation of proper connections transferring load through the diaphragm, from the diaphragm to the wall, down through the wall by means of proper nailing of shear-resisting materials, proper connection of sill plates at the base and, when required, installation of hold-down anchors and proper reinforcing in footings. Inadequate materials or improper connections at any point in the system will limit the shear-resistance capacity of the entire system and will invite failure.

Methodology

Design Steps

Subsequent chapters of Part II take the designer in detailed fashion through the steps necessary for seismic-resistive design of single-family dwellings – the shear walls, the connection of the various structural elements of the house, structural considerations other than shear walls, and some of the more critical non-structural features. The vehicle for the presentation is a series of four model homes, a one-story, a two-story and two versions of split-level construction, models successively more complex but all representative of the problems encountered in seismic design. Because the overwhelming majority of houses are built of wood framing, most of the report's attention is to that type of construction. Many of the design principles apply as well to houses with masonry exterior walls and Chapter II–5 is devoted to a discussion of the similarities and differences in the problems encountered.

Chapter II–2. The first two steps of the methodology are combined in Chapter II–2: Locating Shear Walls and Determining Tributary Areas. The chapter describes the determination of floor and roof diaphragms that require support by shear walls at the edges, including the factor of broken or discontinuous diaphragms, and the chapter presents rules to allow the designer to determine whether other walls in the house should also be used for shear resistance. Once shear walls have been located, the designer must determine what areas of the diaphragms will transfer load to them. In some uniform designs the task is a simple 50-50 proposition, half to the left wall, half to the right. Chapter II–2 describes the simplified cases and also tells how to determine tributary areas in more complicated designs, when more than two parallel shear walls are present or, for example, the new recommendation concerning interior walls that has developed from observation of the damage from the San Fernando earthquake: A multiplying factor applied to interior walls in the first story of two-story houses. Additional tributary area example calculations are given in Appendix B.

Chapter II–3. Having determined the shear walls to be considered and their respective tributary areas of load, the designer is told how to determine the extent of load in Chapter II–3: Determination of Dead Loads and Seismic Loads. As has already been stated, seismic load is a product of tributary area times weight times a seismic factor, with weight being the actual weight of the materials and the seismic factors normally being 0.133 for Zone 3 and half that (0.067) for Zone 2. Seismic Zone maps showing the various parts of the United States included in each of Zones 2 and 3 are given in Part III of the report. No seismic design is stipulated for Seismic Zone 1.

Chapter II–4 and Chapter II–5. When the designer has determined the load to a given line of shear resistance – one wall or more than one – he is ready to design. The procedures for design of shear walls are contained in Chapter II–4 for wood frame walls and in Chapter II–5 for masonry walls. Included are the determination of shear-per-foot as it impacts on the wall under consideration and the comparison with the allowable shear (in tables and graphs) established for the finish material to be used in constructing the wall. Also included are references to appropriate nailing, sill bolt and hold-down anchor schedules to provide overturning resistance and to allow design of effective shear transfer from the shear wall to the floor or foundation.

Chapter II–6. All imaginable design conditions in residential structures could hardly be covered in a single document. An attempt is made to give the designer some insight into major special problems that will occur in the "average" house in Chapter II–6: Miscellaneous Structural Considerations. Included, for example, are commentary on the design of split-level homes, the effects of openings in floor diaphragms for stairwells, the actions of struts and chords, the adequacy of a roof diaphragm for the proposed architectural design, and the efficacy of grades of sheathing and types of nailing for floor diaphragms.

Chapter II—7. Again, though full coverage is not feasible in this document, an effort is made in Chapter II—7: Non-Structural Items to alert the designer to some specific problem areas that result in significant earthquake damage even if a residence is designed with effective shear walls. An adequate shear-transfer system will reduce damaging vibration effects on non-structural features of a house, but some special attention must be given to such items as glazing, fireplaces, veneer, water heaters, floor furnaces, etc.

Calculations

To assist in the step-by-step design methodology, specially devised Calc Forms are incorporated in this report and their use is described by example in Chapters II—3 and II—4 discussing the model homes. Blank copies of the Calc Forms are also presented in conjunction with appropriate sections of the Design Methodology in Part III. It is suggested that the blanks may be duplicated on an office copier for the designer's future use.

Use of the Report

It was noted earlier that the methodology presented in terms of the model homes covers most facets of seismic-resistive design from the simplest to the highly complicated residential dwelling. A designer faced with a simple one-story house with sloped roof and level ceiling, for example, may be able to study only the opening remarks of Chapters II—2, II—3, II—4 and II—5 plus the comments on the Model 'A' home before starting to design. For a full grasp of the methodology, however, it is recommended that all model homes be studied through each chapter. Readers are cautioned, for instance, that some particular conditions such as walls other than 8'-0" nominal height, sloping and/or exposed-beam ceilings, and the special garage front wall detail are covered only in the discussion of the Model 'D' home.

Development of a seismic-resistive design for a split-entry home is included as Appendix A.

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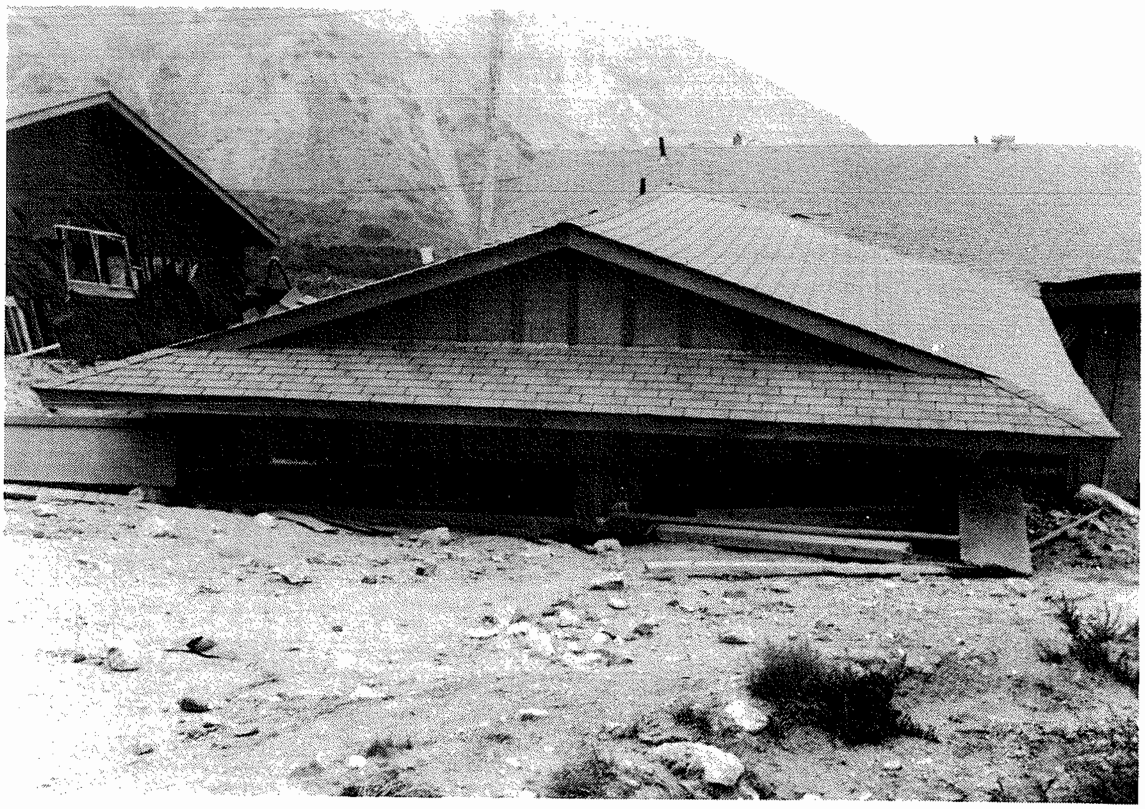

CHAPTER II–2 LOCATING SHEAR WALLS and DETERMINING TRIBUTARY AREAS

Introduction

This chapter discusses the first two steps in the design of a residence for resistance to earthquake lateral forces: (1) Locating shear walls properly; (2) Determining tributary areas of diaphragms contributing load to the shear walls. This and subsequent chapters of Part II introduce through practical application the Recommended Design Methodology set forth in Part III. Provisions concerning location and length of shear walls, though they leave the designer considerable latitude, are nevertheless presented as recommended requirements. As for tributary area, in all cases all roof and floor areas should be assigned to some line of shear resistance but it is recognized that design conditions may prevail that require judgmental decisions to satisfy the intent of the provisions and, for that reason only, the provisions on tributary area are called guidelines rather than requirements.

The practical application is accomplished through discussion of four model homes, A, B, C and D for which calculations and design are presented according to the Recommended Design Methodology given in Part III. The example houses are shown schematically in

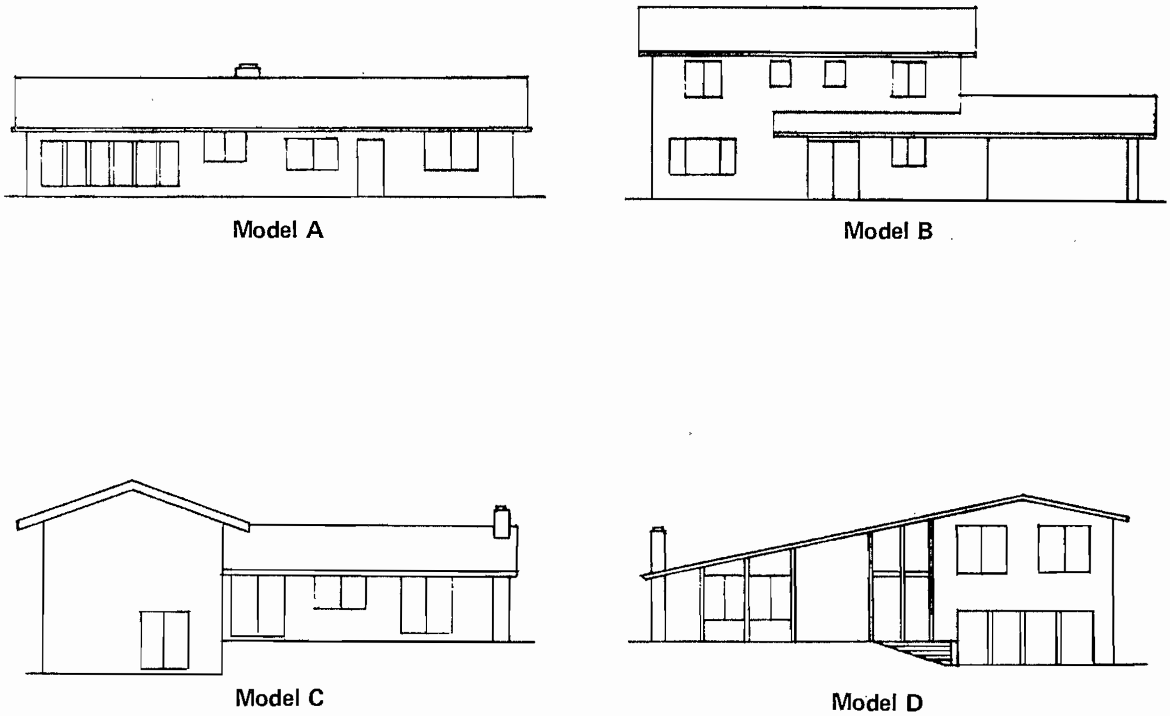

FIGURE 2.12. Model Home Elevations (Schematics).

Figure 2.12. The elevations, roof plans and floor plans are shown with the detailed discussion of each example. Model A is a simple one-story house, Model B is a two-story and Models C and D are versions of split-level design. (Other floor plans are presented as examples in Appendices A and B.)

Each model house is constructed of conventional wood framing and standard finish material; together they represent much of the current residential construction and many of the problem areas encountered in seismic-resistive design. Houses with floor plans similar to the examples, particularly Models B and C, were subjected to severe damage in the San Fernando earthquake.

Locating Shear Walls

Section 2 of Part III* defines the recommended conditions that must be met for a section of wall to be considered as a shear wall and indicates the required locations and lengths for such walls.

Materials that may be used to develop shear resistance are 1" diagonal sheathing, gypsum sheathing board and wallboard, fiberboard, stucco, gypsum lath and plaster, hardboard and plywood.

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* Section I of Part III contains definitions applicable to the publication.

For a section of wall to qualify as a shear wall it must be at least 4'-0" in length, must meet height-to-width ratio requirements dependent on the material used, and, when applicable, certain limitations regarding interruption of the shear-resisting material for hardware, etc. All materials other than plywood are limited to a maximum height-to-width ratio of 2:1; a 4'-0" long wall, unless sheathed with plywood, may be used only with nominal 8'-0" high construction. If wall height is 9'-0", for instance, the least length of non-plywood wall that may be considered as acting is 4'-6" since 9'/2 = 4.5'.

Any interruption of the shear-resisting material is considered to be a hole. Small holes are allowed; very small holes, such as those created by a pipe or by an electrical wall switch, are ignored. Since a wall's total shear resistance is equal to the rated shear-per-foot of its material times the wall's length, the width of larger holes (as described in Section 2.1A3 of Part III) should be subtracted from the length in determining the allowable load the wall can resist.

Section 2 of Part III also specifies that shear walls be located along all lines of exterior walls and that interior walls that extend to the roof be shear walls. Among other critical loactions where walls are required to be considered as shear walls are interior one-story walls when the house has a flat roof or open-beam or cathedral ceiling and, in a requirement that goes beyond current standard practice, certain interior walls of the first story of two-story construction.

Although the Design Methodology does not specifically say so, not every eligible wall along a line of resistance need be considered as acting. As a matter of practicality – because the more wall length used, the lower the shear-per-foot – all eligible walls are almost always used. The important considerations, however, are that adequate shear resistance is provided for the loads developed and that resistance is provided at all edges of each separate diaphragm.

As an important corollary, in two-story construction the second floor must not exceed a 1-1/2:1 diaphragm ratio because diaphragms are not designed by the methodology used in this report. Apart from these considerations, it is not critical if an otherwise eligible wall is not used in the design since all remaining wall will still be designed for the total load.

Determining Tributary Area

Section 3 of Part III provides guidelines for determination of tributary area contributing load to each line of shear resistance with particular emphasis on tributary widths, lines acting together and the adjustment in area required for interior first-floor shear walls in two-story residences.

Simply stated, the tributary width to a given shear wall is one-half the distance between it and the next adjacent parallel shear wall, similar in principle to the condition expressed in the discussion of the steel beam in Chapter II–1 (Figure 2.8) with one important difference. A uniform load across the steel beam was assumed; in seismic analysis the same assumption is made even though the load between shear walls is not uniform and, indeed, when the diaphragm is not even of constant length. It is nevertheless assumed that all load developed on each side of the centerline between walls is distributed to that wall on the same side of the centerline.

When shear walls occur on each side of the wall whose tributary width is to be determined the width is one-half the distance between the walls to either side.

In the case of cantilevered diaphragms, the total area of cantilever must be taken by the line of resistance (or lines acting together) at the cantilever support.

As was noted in the discussion on locating shear walls, field observations following the San Fernando earthquake indicated that some first-floor interior shear walls of two-story houses received more load than would be anticipated by the usual method of figuring tributary widths. A multiplying factor is introduced in Part III which must be applied to all such walls. (Further discussion on this is included in the comments on Models B, C and D.)

In calculating tributary widths, exactness would dictate that first the clear distance between walls should be determined, then halved, then the wall width added. In view of the various assumptions necessary with regard to loads, such exactness may be disregarded. Architectural plans usually are dimensioned from outside face of exterior wall to centerline of interior wall. Halving the dimensions indicated on the plans provides sufficient accuracy. Use of the exact method will preclude difficulty when a particular dimension includes the total width of wall at one side and excludes it at the other. The exact method is used in the model-home examples in this chapter but that should not be interpreted to mean the exact method must be used in practice. Tributary area plans in this chapter present all dimensions as they would appear on the architectural floor plan. Decimal dimensions such as 12.33' are used to designate all tributary widths in order to differentiate them from the architectural dimensioning.

Familiarity with how to figure tributary area is most important. The discussion will develop that, with experience, a designer may determine that certain shear resistance lines are more than sufficient "by inspection," and that only one or two need be calculated. This possible experience-based "shortcut," most likely for one-story construction, enhances design efficiency and economy but it also makes it essential that the tributary areas for critical walls that are to be calculated are calculated correctly.

Model Homes

In the discussions of each of the example homes to follow, the pertinent Sections of Part III are referenced with respect to location of shear walls and determination of tributary area. It is suggested that the reader scan Sections 2 and 3 of Part III before continuing this chapter, then study the referenced sections or subsections in Part III as they are noted in the discussions of the model homes.

MODEL A

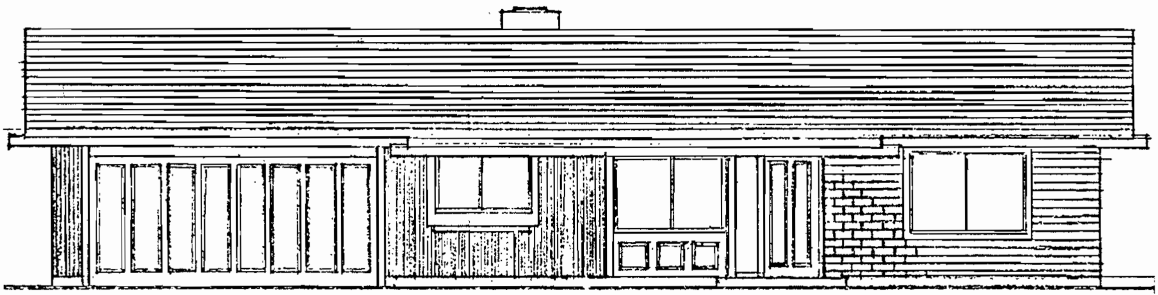


FIGURE 2.13. Floor Elevation – Model A.

The one-story house used as Model A is a fairly simple residence—1288 square feet with another 550 square feet in the oversized two-car attached garage.

Finish materials and framing:

Roofing	Wood shake
Sheathing	Spaced
Roof framing	Prefabricated wood trusses
Ceiling	Gypsum board with acoustic finish
Exterior wall finish	Plywood and brick veneer
Interior wall finish	Gypsum board

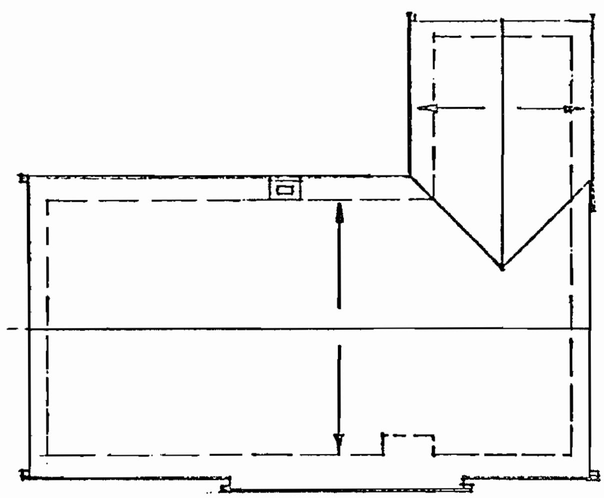


FIGURE 2.14. Roof Plan – Model A.

Design sections in Part III applicable to locating shear walls and determining tributary areas:

2.1A	2.1B1	2.1B2a
3.1	3.3A1	3.3A3

Model A demonstrates the simplest example for locating shear walls and determining tributary areas and, in addition, indicates alternatives available for the design of wings.

Inspection of the floor plan (Figure 2.15) reveals the available shear wall along all exterior wall lines. Even if the alternative rear wall for the living-dining area were chosen, the rear wall along the garage would still provide considerable shear resistance along that line. The front wall of the house appears to provide the shear resistance required. In the master bedroom area there seems to be sufficient wall along the line containing the sliding glass

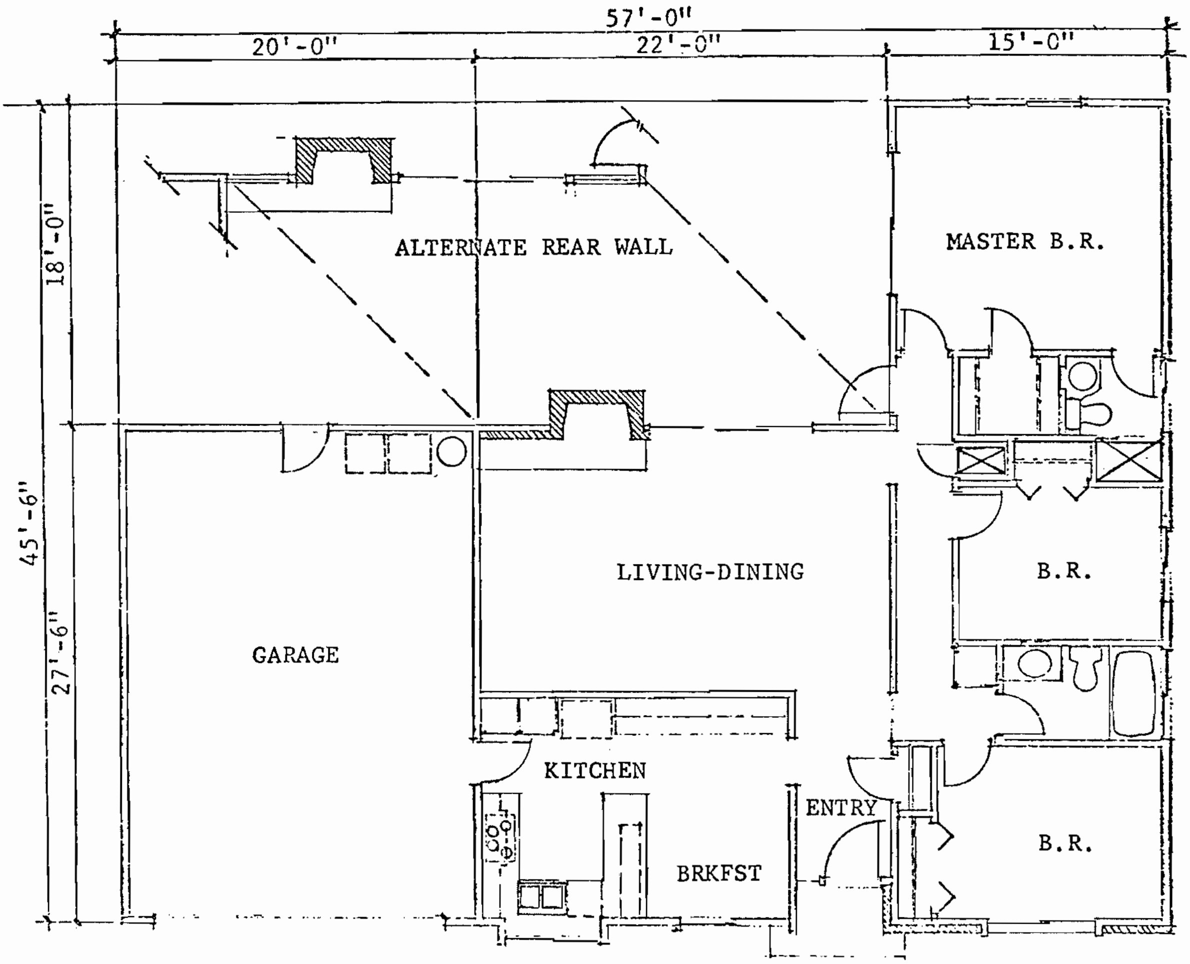


FIGURE 2.15. Floor Plan — Model A.

door to resist the lateral load generated by that small wing. The house would seem to present no particular problems for design.

One potentially serious problem is indicated in the Model A design, having to do with the recommendation that there be solid wall at all corners of a residence because solid, interconnected walls at corners tend to resist the torsional effect of earthquake motion. As can be seen in the Floor Plan above, the door to the outisde from the hallway to the master bedroom eliminates wall at a critical point, as would the alternative rear wall that would eliminate the wall just around the corner from the hallway door. When architectural planning does not allow for a shear wall of at least 4' in length it is advisable to locate as much wall as possible at a corner, such as was done adjacent to the sliding door of the master bedroom.

For Model A, all shear walls are easily visualized, as indicated by the solid wall sections in Figure 2.16. (It is assumed that the wall between the garage and the main portion of the house extends to the roof and therefore is considered to be a shear wall.)

Two alternative tributary area diagrams for earthquake motion in the transverse direction are given in Figures 2.17 and 2.18. In the tributary area diagrams, the earthquake motion

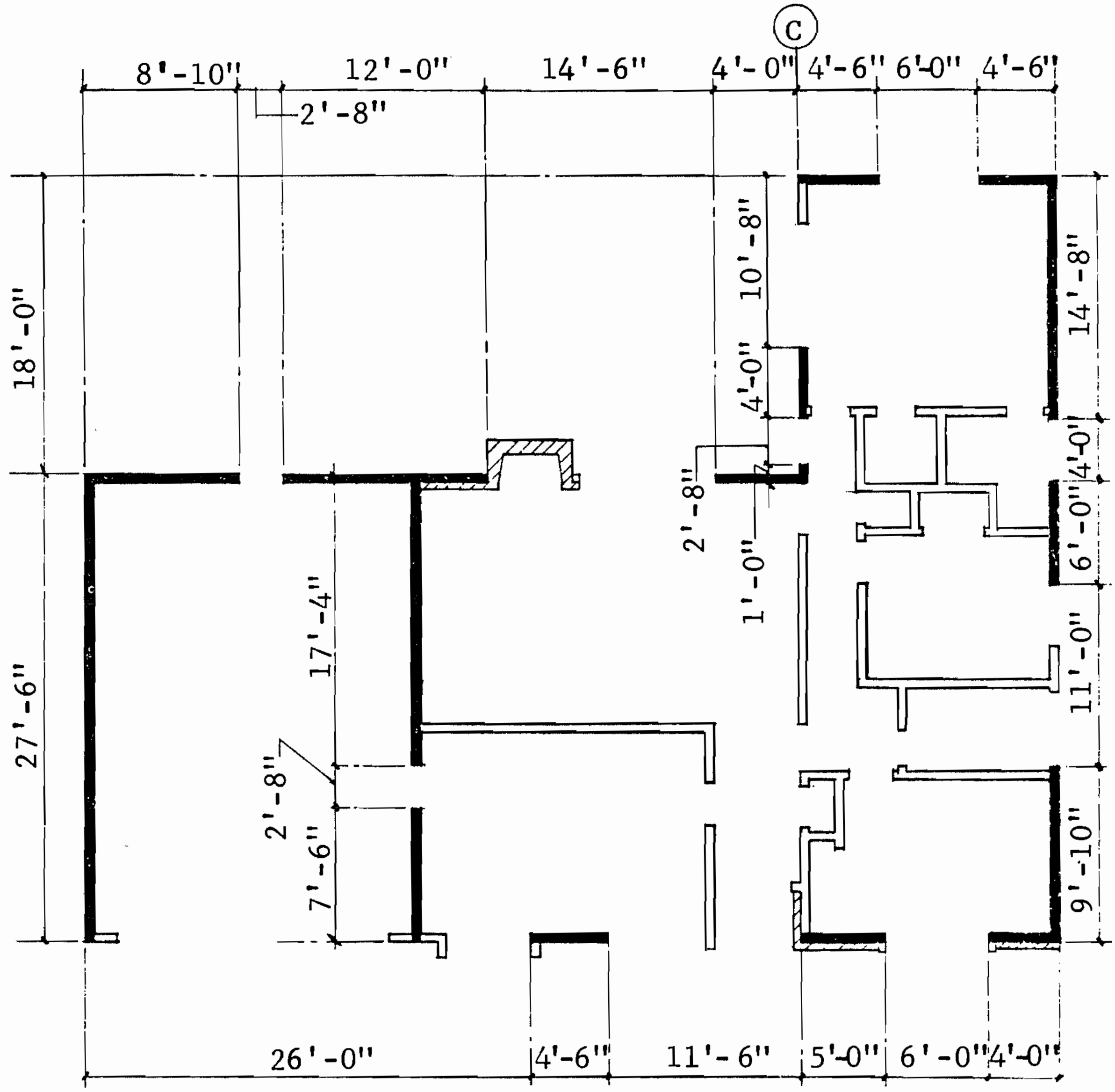


FIGURE 2.16. Shear Wall Plan — Model A.

direction is shown with the bold double arrow; the chevrons depict the area tributable to shear walls. Specifically, the area represented by one-half of a chevron is tributable to the adjacent shear wall. This can be readily seen in the wing portion of Figures 2.17 and 2.18.

Tributary Widths — Figure 2.17.

The possibility of using an interior wall as a shear wall is illustrated in Figure 2.17, considering the wall between the living-dining area and the hallway. That wall is an extension of the exterior wall on Line C and it would be expected that the top plates would be continuous from the front to the rear of the building, providing strut action to the two interior wall segments. This type of design would also require a strut to be developed from the main roof to the wall because otherwise all shear would have to be transferred through the portion of the diaphragm over the wing.

A to B Since both shear walls are included in dimension from A to B,

$$\frac{20'\text{-}0''}{2} = 10'\text{-}0''$$

B to C Neither wall included $\frac{22'\text{-}0''}{2} = 11'\text{-}0''$

C to D Both walls included $\frac{15'\text{-}0''}{2} = 7'\text{-}6''$

Tributary width:

Line A	10.00′
Line B	10.00 + 11.00 = 21.00′
Line C	11.00 + 7.50 = 18.50′
Line D	7.50′

<u>Tributary Widths – Figure 2.18.</u>

Per Section 3.3A3, the shear wall on Line C at the exterior-interior corner wall of the wing need not be considered as shear wall for the main body of the house. As is demonstrated in Figure 2.18 the only area tributary to that wing wall is the portion of the roof adjacent to the line of resistance. By distributing the tributary area to the walls as shown in Figure 2.18 the interior wall on Line C need not be developed as a shear wall and the loads, as represented by the areas, are going to the locations where the longest lengths of wall exist. It is this method that is used in the calculations in the subsequent chapters.*

A to B Since both shear walls are included in dimension from A to B,

$$\frac{20'\text{-}0''}{2} = 10'\text{-}0''$$

B to D $\frac{22'\text{-}0'' + 15'\text{-}0'' - 4''}{2} = 18'\text{-}4''$ clear

C to D $\frac{15'\text{-}0''}{2} = 7'\text{-}6''$

Tributary width:

Line A	10.00′
Line B	10.00 + 18.33 = 28.33′
Line C	7.50′
Line D	18.33 + 0.33 = 18.67′ and 7.50′ at wing

*Calculations of tributary width are not shown for the other model homes when tributary width is simply 50% of the distance between shear walls with no further adjustment needed.)

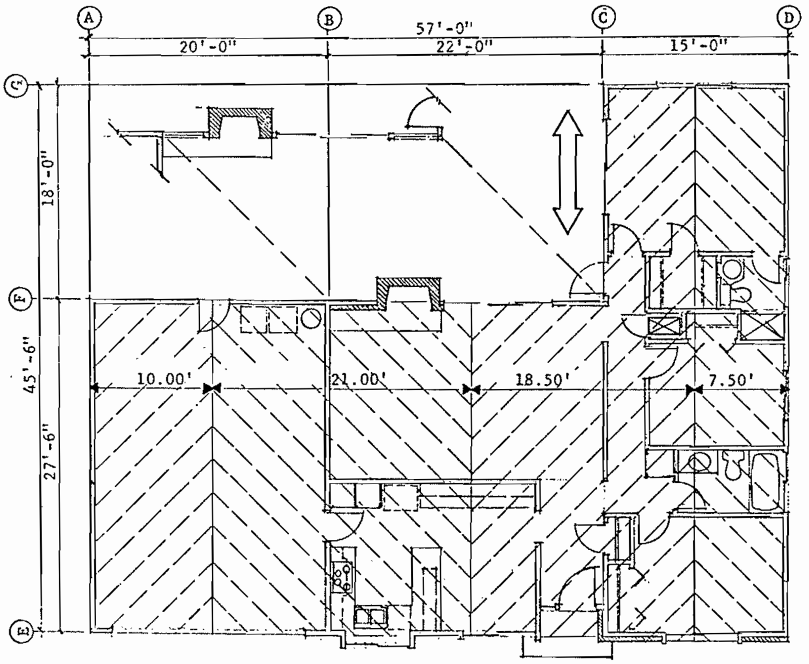


FIGURE 2.17. Tributary Areas, Transverse Direction — Alternative 1, Model A.

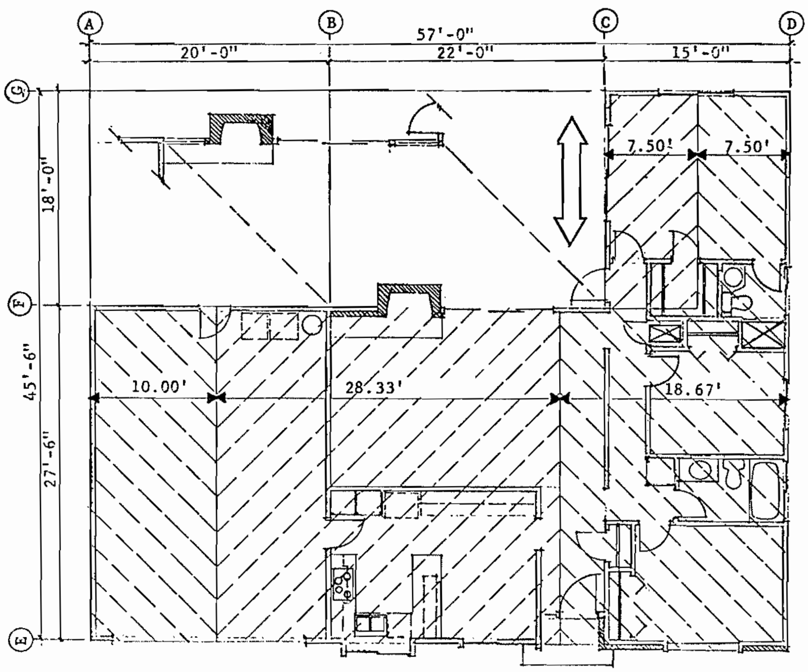


FIGURE 2.18. Tributary Areas, Transverse Direction — Alternative 2, Model A.

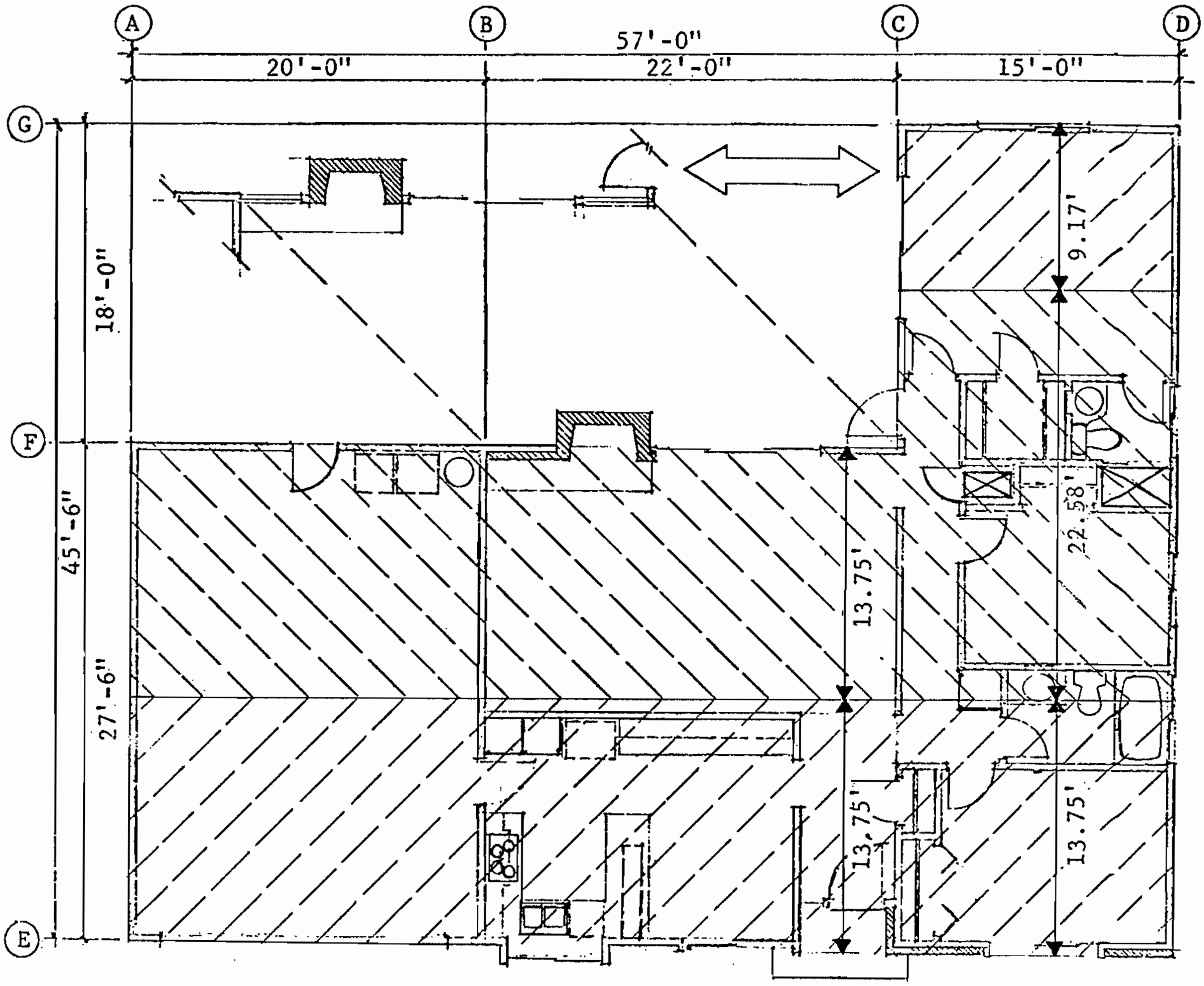


FIGURE 2.19. Tributary Areas, Longitudinal Direction – Model A.

Tributary Widths – Figure 2.19.

For loads in the longitudinal direction, as shown on Figure 2.19, the determination of tributary widths is also quite simple since the only lines of shear resistance are at the three lines of the exterior walls of the house (E, F and G).

E to F	Both walls	$\frac{27'\text{-}6''}{2} = 13'\text{-}9''$
F to G		$\frac{18'\text{-}0'' - 4''}{2} = 8'\text{-}10''$ clear

Tributary width

Line E	13.75′
Line F	13.75 and 13.75 + 8.83 = 22.58′
Line G	8.83 + 0.34 = 9.17′

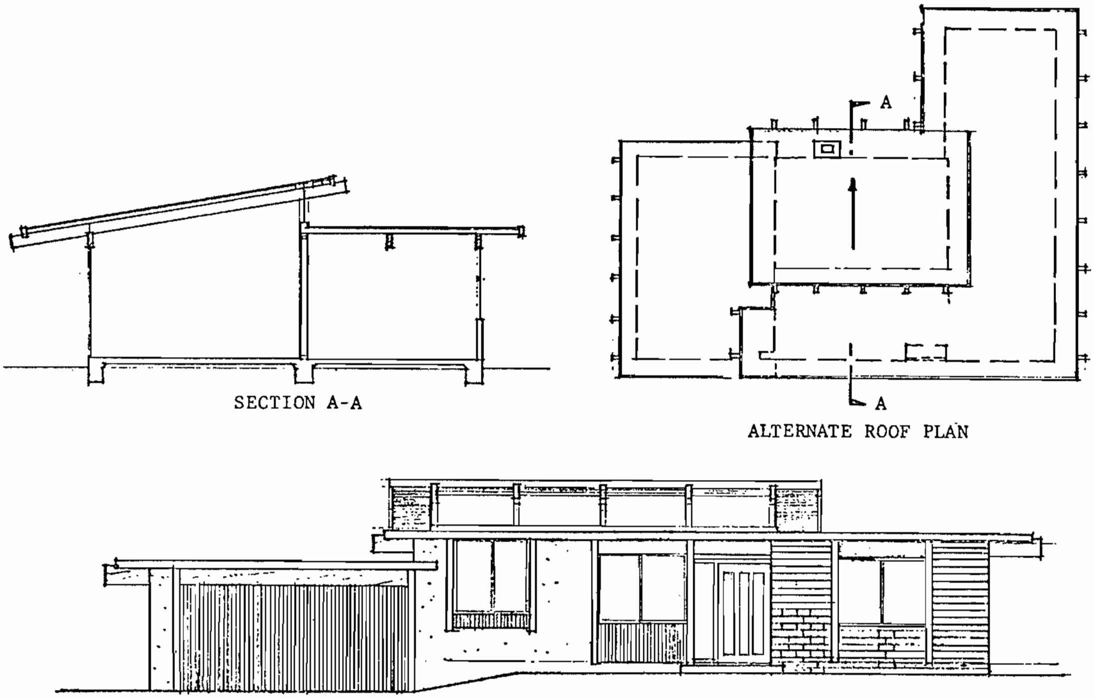


FIGURE 2.20. Alternative Front Elevation – Model A, depicting design problems not solvable by the provisions of this publication.

The design of "no-problem" Model A house can be altered to become unworkable under the provisions of this report. Figure 2.20 shows the garage lowered so that the shear wall at the rear cannot have the load from the rear of the house transmitted to it by strut action—the top plates would be discontinuous at the junction of garage and house. If the alternative rear wall were used, there would be no shear wall to support the high portion of the roof over the living-dining area. In addition, with glass as shown in a clerestory on the front elevation below the elevated roof, no shear wall would be present to resist the load at the edge of the separate roof diaphragm. In fact, the house would consist of three separate portions, each with its own roof diaphragm, and therefore would require shear walls along all four edges of each diaphragm to properly support all elements. If some shear wall were added at the clerestory, if the alternative rear wall were not used, and if appropriate interior walls were used to support diaphragm edges, the Figure 2.20-house could be designed by the provisions of this report. Otherwise, design to resist lateral load would require engineering analysis.

MODEL B

FIGURE 2.21. Front and Rear Elevations – Model B.

Model B is a 2016-square-foot, two-story house with an attached garage of approximately 500 square feet. Finish materials and framing:

Roof	**Roofing**	**Asphalt shingle**
	Sheathing	**3/8″ plywood**
	Roof framing	**Standard wood framing**
	Ceiling	**Gypsum board**
Second Floor	**Flooring**	**Carpet**
	Underlayment	**None**
	Sheathing	**5/8″ plywood**
	Framing	**Wood**
	Ceiling	**Gypsum board**
Exterior Wall Finish		**Hardboard and Veneer**
Interior Wall Finish		**Gypsum board**

Design sections in Part III applicable to locating shear walls and determining tributary areas:

At first story:	**2.2A2**	**2.2B2**	**2.3D**		
	3.1	**3.2A**	**3.3B1**	**3.3B2**	**3.3B3**
At second story:	**2.2A1**	**2.2B1**			
	3.1	**3.3A1**	**3.3A4**		

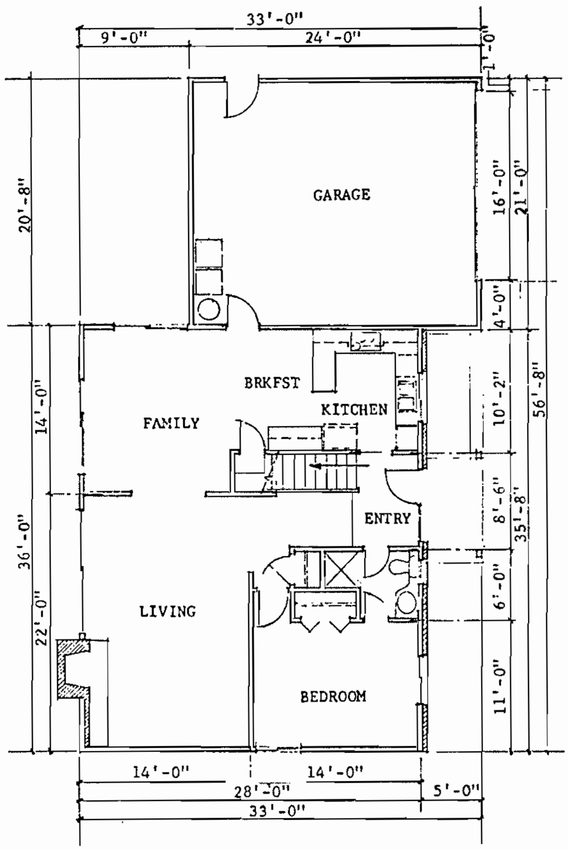


FIGURE 2.22B. First Floor Plan – Model B.

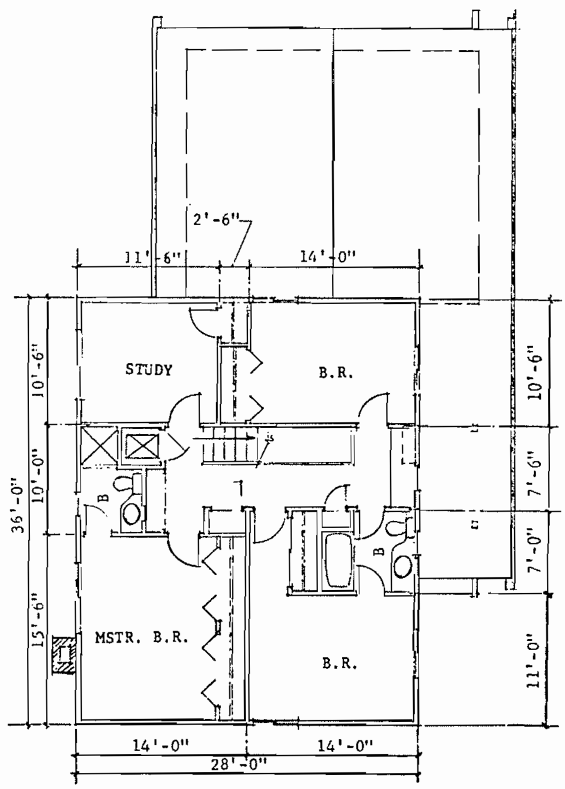


FIGURE 2.22A. Second Floor Plan – Model B.

Plans for the first and second floors are given in Figure 2.22. Note the roof over the front entry; it will receive special comments later.

Many features of Model B, such as floor plan, attached garage with service facilities, and large amounts of glass, are features similar to those in houses damaged in the 1971 San Fernando earthquake. Though such conditions may pertain only primarily to warmer climates, they serve to point up a common principle of seismic design: Houses with more solid exterior wall will require fewer special requirements, such as hold-down anchors and closer sill bolt spacing.

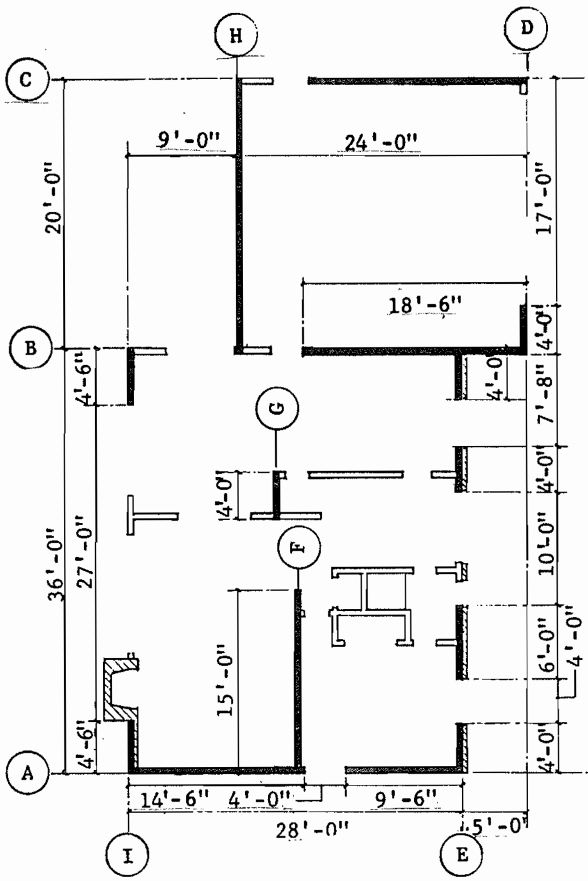


FIGURE 2.23. Shear Wall Plan — First Floor, Model B.

The shear wall plan for the first floor of Model B is given in Figure 2.23.

The interior walls of the second floor are not considered as shear walls since they do not extend to the roof level; only the exterior walls of the second floor are shear walls and a shear wall plan is not included.

Tributary area diagrams for earthquake motion in the longitudinal direction are given in Figure 2.24.

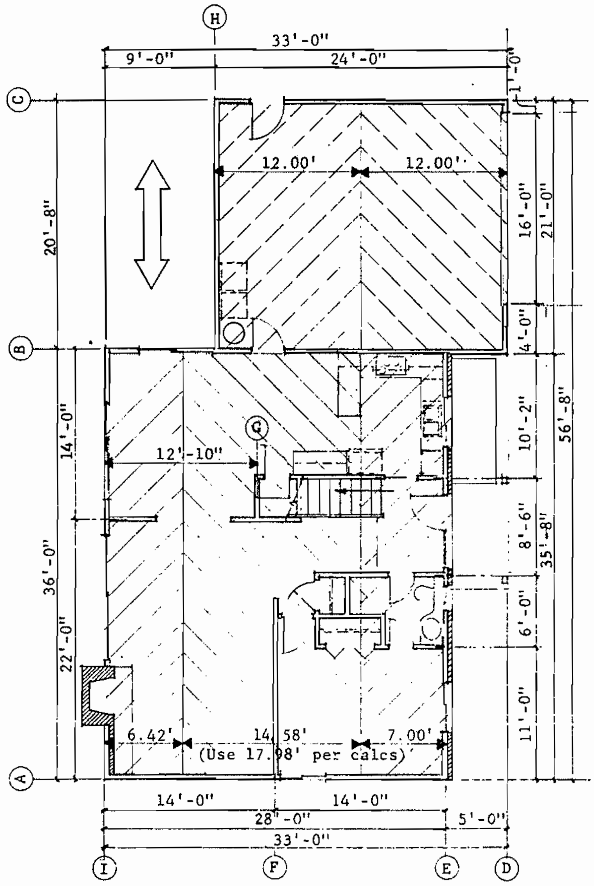


FIGURE 2.24A. Tributary Areas, Longitudinal Direction – First Floor, Model B.

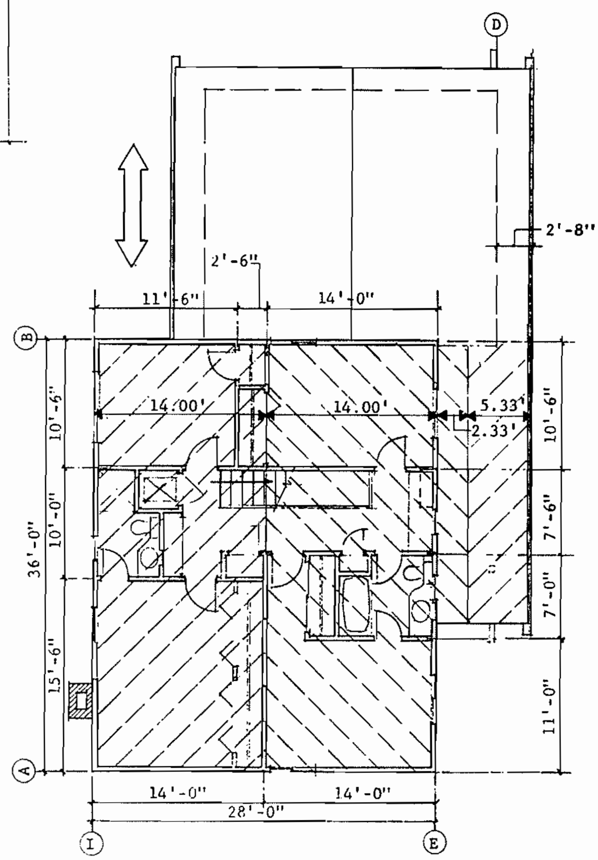


FIGURE 2.24B. Tributary Areas, Longitudinal Direction – Second Floor, Model B.

Tributary Widths – Figure 2.24A

The tributary area to the first floor interior shear walls (Lines F and G) for load in the longitudinal direction offers the first example of the use of the multiplying factor. The 4'-0" shear wall on Line G is within 3'-0" of the 15'-0" wall on Line F and the walls must be considered as acting together. When considered to be acting together they are 19 feet in length, longer than the total length of shear wall at either the front or the rear. Since the design methodology (Table 3.1) exempts only walls shorter than the exterior walls, this wall (F and G) must be considered as shear wall.

E to F	$\frac{14'\text{-}0''}{2} = 7'\text{-}0''$
F to G	14'-0" – 12'-10" = 1'-2" center to center
G to I	$\frac{12'\text{-}10''}{2} = 6'\text{-}5''$
Tributary width:	Line E 7.0'
	Line I 6.42'
	Lines F and G 7.0 + 1.17 + 6.41 = 14.58'

Multiplying Factor (Section 3.3D3) $\frac{L_e}{L_i} = \frac{9.0}{19.0} = 0.47$

Multiplying Factor = 1.25 (since L_e/L_i is greater than 0.30)

Total width to consider, in accordance with Section 3.3B3b

$$= 1.25\,(14.58 - 1.17) + 1.17$$
$$= 16.76 + 1.17 = 17.93'$$

Tributary Widths – Figure 2.24B (Front covered porch only)

The porch roof at the front of the house contributes load to the first-floor shear walls at Line E as well as to the wall at the front of the garage (Line D). The 2'-8" cantilever portion of the porch roof is assigned entirely to Line D with the remainder of the porch roof divided equally between Lines D and E.

D to E	5'-0" – 4" = 4'-8" clear total
Tributary width:	Line D $\frac{4.67}{2} + 0.33 + 2.67 = 5.33'$
	Line E $\frac{4.67}{2} = 2.33'$

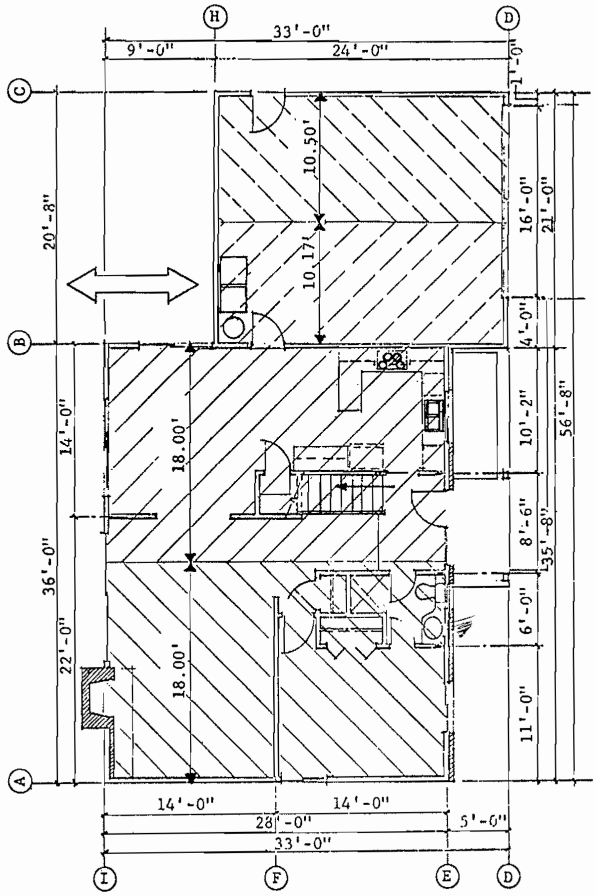


FIGURE 2.25A. Tributary Areas, Transverse Direction – First Floor, Model B.

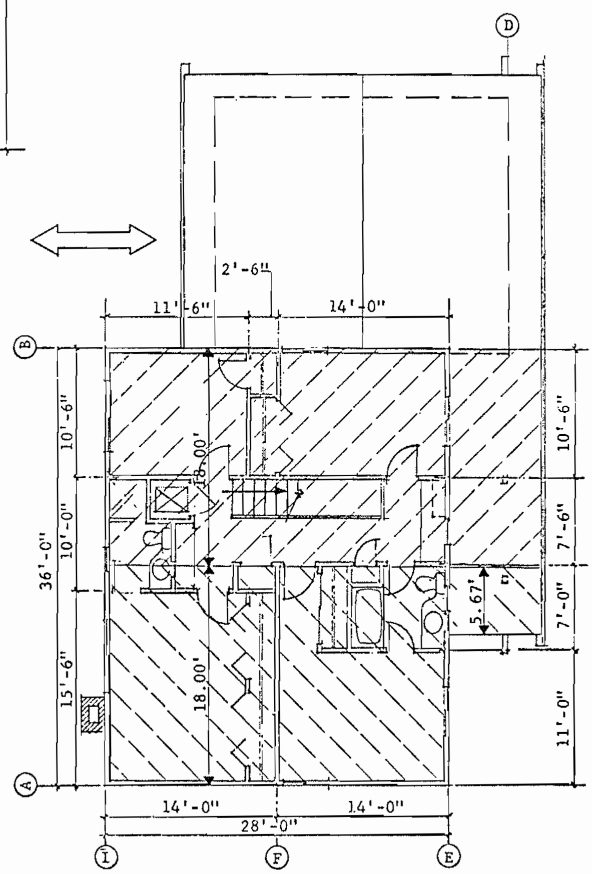


FIGURE 2.25B. Tributary Areas, Transverse Direction – Second Floor, Model B.

Tributary area diagrams for motion in the transverse direction for the first and second floors of Model B are given in Figure 2.25.

Figure 2.25A

Diaphragm ratio	$\frac{36.0}{28.0}$ = 1.29:1 is less than 1.5:1
Shortest exterior shear wall	18'-6" (Lines A and B)
Maximum interior wall length, per Table 3.1	18.50 x 0.75 = 13.875' (shorter walls need not be designed)
Effective length to right of stairs	8'-0"
Effective length to left of stairs	9'-4"

Neither wall need be designed, see below

The "hole" in the diaphragm for the stairway is 12'-6" long. Since that is less than one-half the diaphragm depth of 28'-0" no shear wall need be located adjacent to the stairs to meet the Design Methodology. The only other reason the walls to either side of the stairs might need to be designed would be because of their length. Since they are more than 3'-0" apart they may, but need not be, considered as acting together. If each acts separately, each is shorter than 13.875' and therefore need not be considered as a shear wall. Since the walls at each end seem sufficiently long, adding these short segments of interior wall would probably result only in additional construction expense; the short segments are therefore not considered.

Figure 2.25B. (Front covered porch only)

The porch roof is the type of small diaphragm that can mislead the designer. As shown in Figure 2.25B the only shear walls for the main body of the residence are at Lines A and B, and all load to the right of the line midway between A and B is taken to Line B and all load to the left is taken to Line A. As shown on the front elevation (Figure 2.21) the porch roof frames into the second-floor stud wall slightly above the level of the second floor itself. The load from the porch roof should therefore be considered along with the second-floor diaphragm loads and does not enter into consideration of loads for shear walls above the second floor.

Total width of porch roof	23'-8"	
	Line A	23'-8" – 18'-0" = 5'-8" = 5.67'
	Line B	18.0'

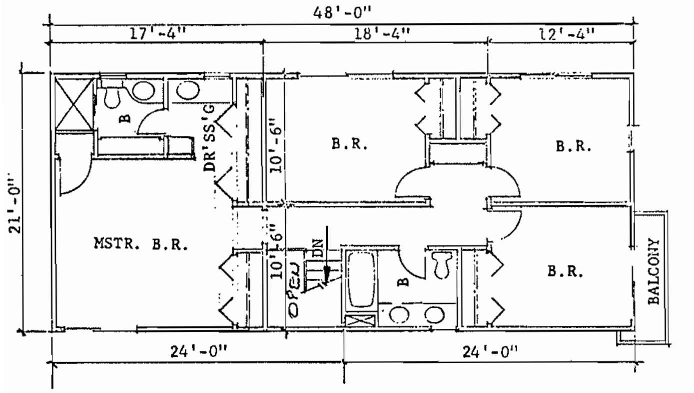
MODEL C

FIGURE 2.26. Front and Rear Elevations – Model C.

Split-level houses come in many configurations. The house shown in Figure 2.26 is typical of split-levels in many parts of the country. The second-floor wall front elevation is intended to convey a picture of solid wall above and below the windows with only a change in finish material indicated. Finish materials and framing:

Roof	Roofing	Asphalt shingle
	Sheathing	1/2" plywood
	Roof framing	Prefabricated wood trusses
	Ceiling	Gypsum board
Second Floor	Flooring	Carpet
	Underlayment	5/16" particleboard
	Sheathing	1/2" plywood
	Framing	Wood
	Ceiling	Gypsum board
Exterior Wall Finish		Fiberboard and Wood siding
Interior Wall Finish		Gypsum board; 5/16" plywood in family room

First and second floor plans and roof plan for Model C are given in Figure 2.27.



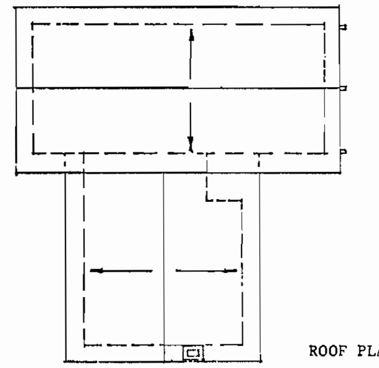


FIGURE 2.27A. Second Floor Plan – Model C.

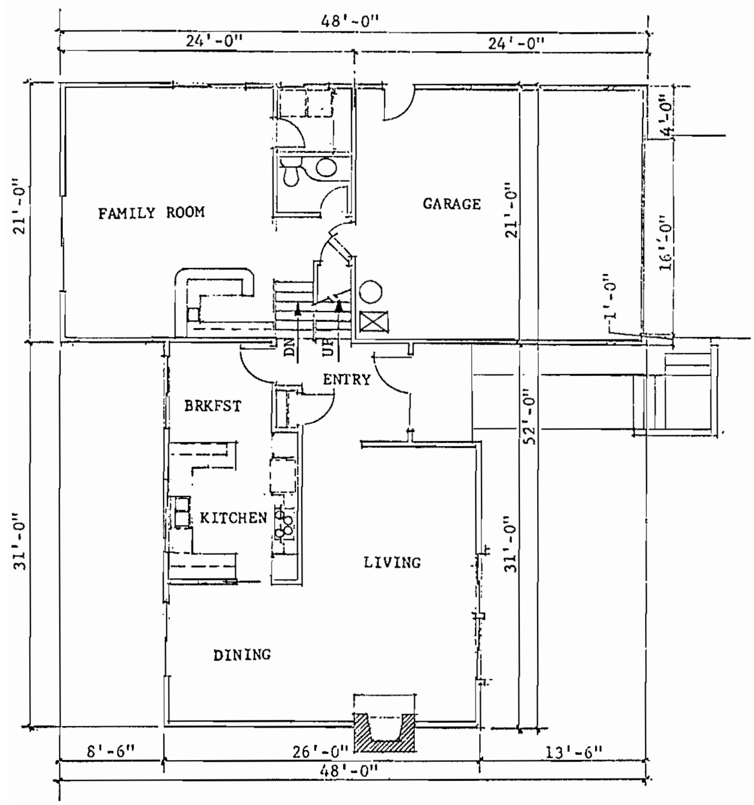


FIGURE 2.27B. First Floor Plan – Model C.

Design Sections in Part III applicable to locating shear walls and determining tributary areas for Model C:

General:	**2.3A1b**	**2.3A2**	
	3.1		
At first story:	**2.2A2**	**2.2B2**	**2.2B3**
	3.3B1	**3.3B3**	
At second story and middle level	**2.1A**	**2.1B1**	
	2.2A1		
	3.3A1		

The shear wall plan for the first floor of Model C is given in Figure 2.28. A shear wall plan for the second floor is not given because none of the interior walls in the second floor extends to the roof and therefore only the exterior walls at the second floor are shear walls.

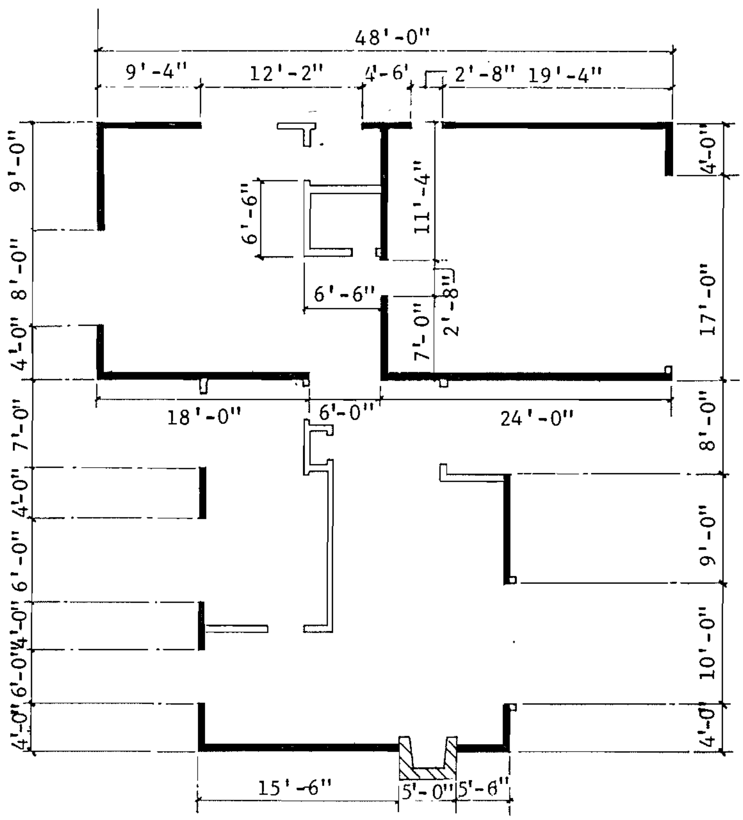


FIGURE 2.28. First Floor Shear Wall Plan – Model C.

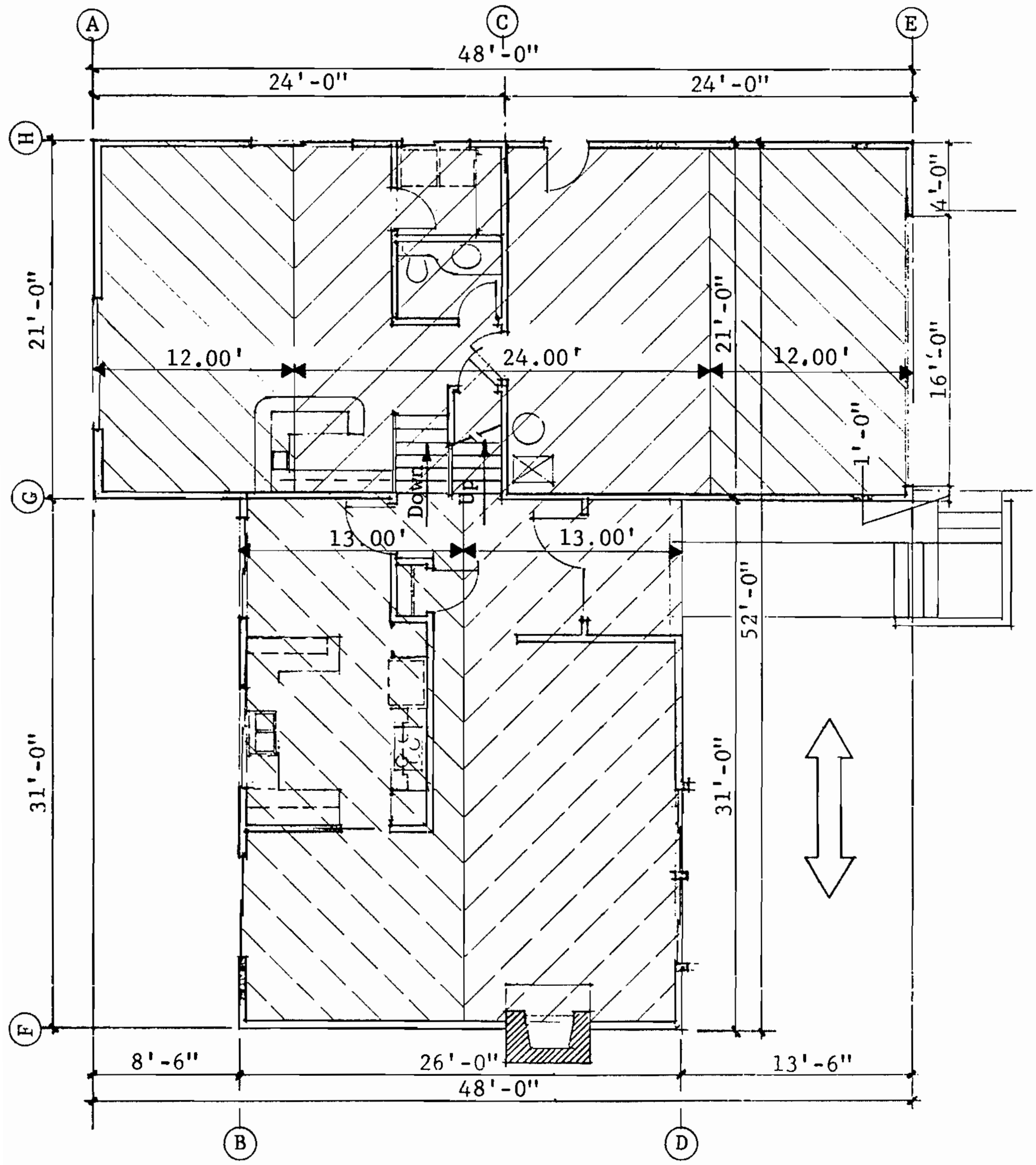


FIGURE 2.29. Tributary Areas, Longitudinal Direction – First Floor, Model C.

For motion in the longitudinal direction (Figure 2.29 above) the tributary areas for all portions of Model C are simply 50% of the distance between exterior shear walls with no adjustments necessary. No longitudinal second floor plan is shown for that reason.

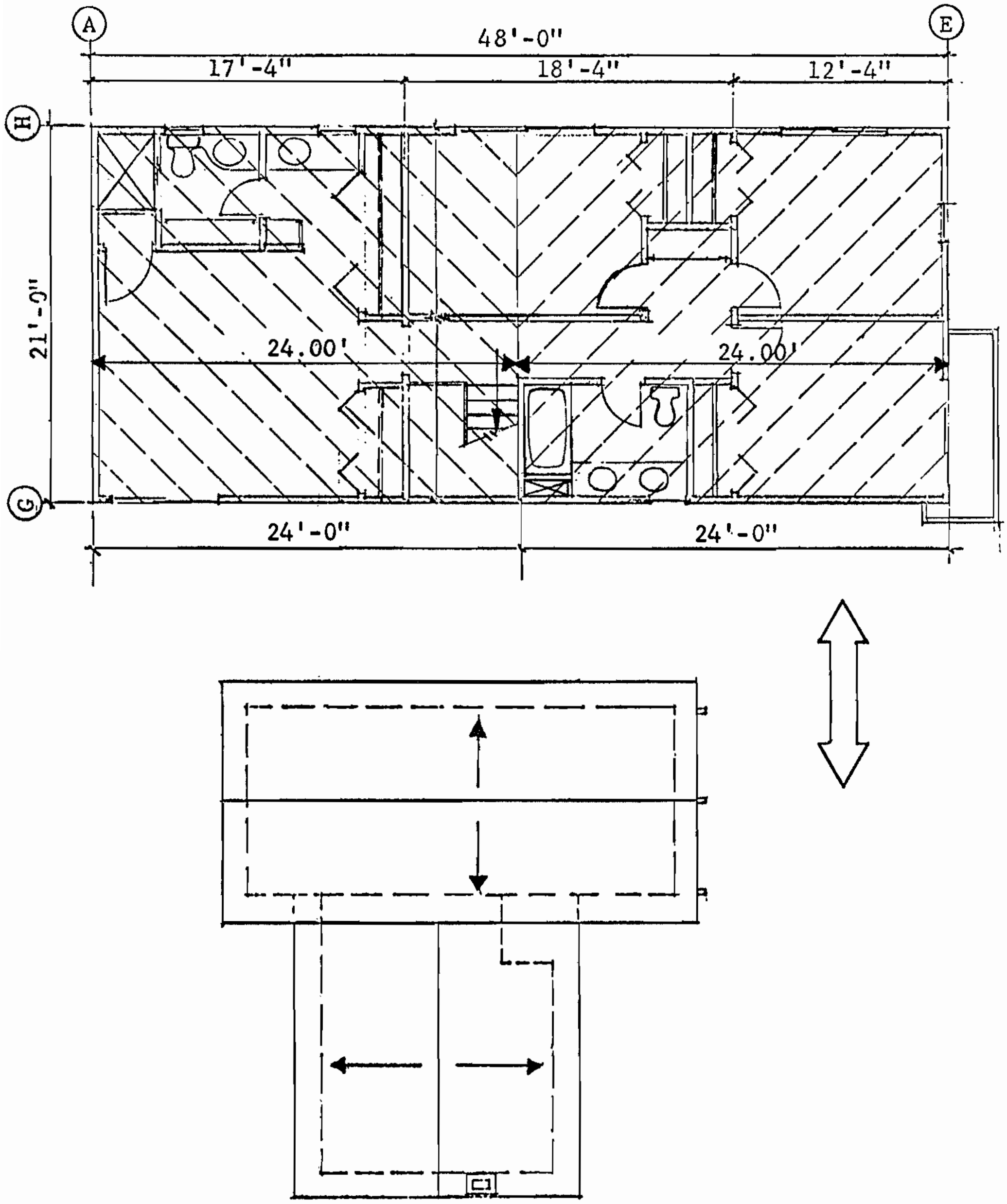


FIGURE 2.30A. **Tributary Areas, Transverse Direction — Second Floor, Model C.**

For motion in the transverse direction, the tributary areas are shown in Figures 2.30A and 2.30B. The first floor interior wall of the two-story portion must be considered for loads in the transverse direction in the same manner as for Model B. Since the walls to the

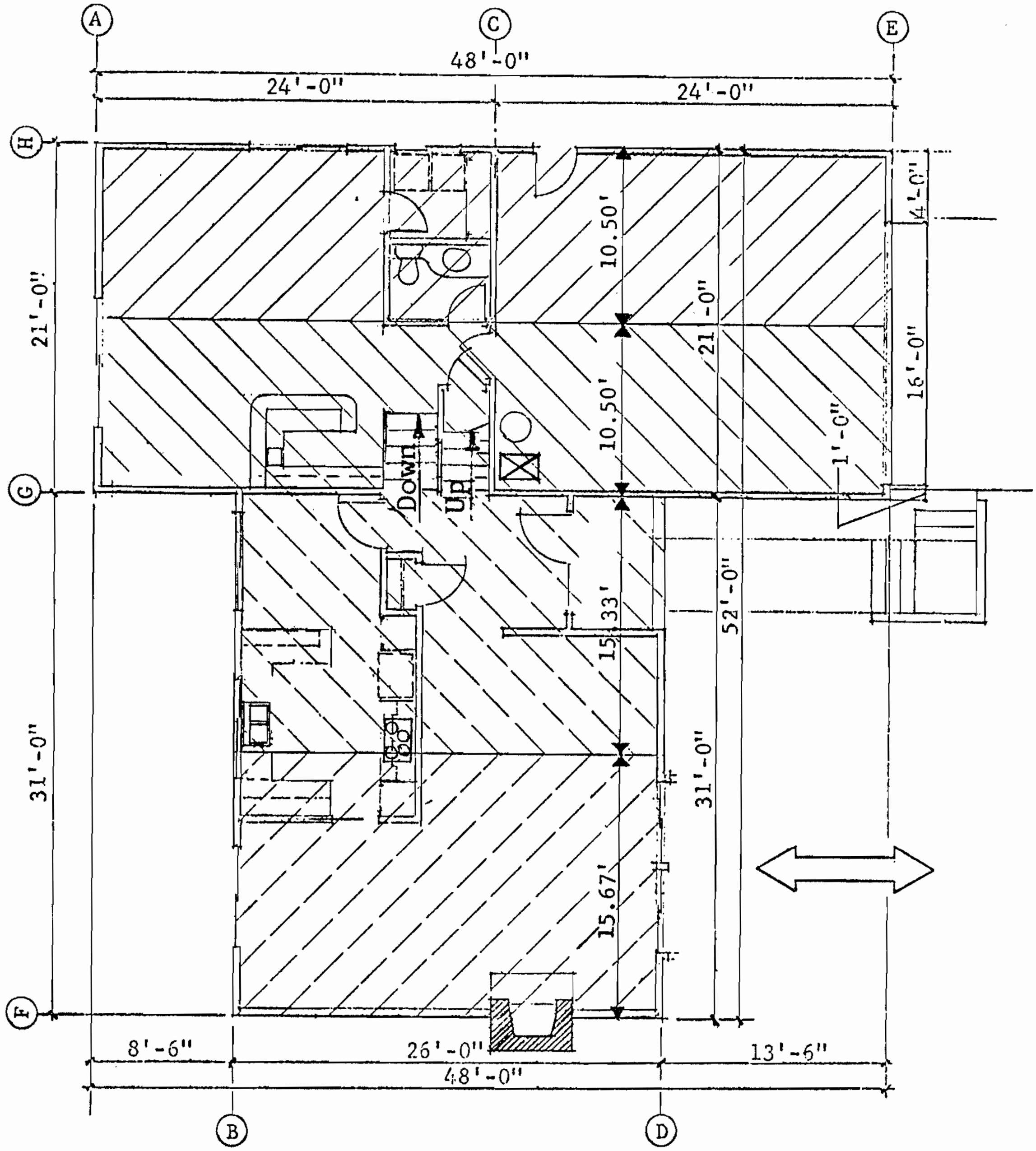


FIGURE 2.30B. Tributary Areas, Transverse Direction – First Floor, Model C.

left of Line C are but 6'-6" from this line (shown in Figure 2.28), these walls could be considered as acting together with the walls on Line C. Total effective length of shear wall along this line would be 6'-6", which is less than 1/2 of the wall length along Line C. Additional tributary area would be created by including this wall, so, because of its length, it has been omitted from consideration as allowed by Design Section 2.2B2b. Note: That design section requires that a wall be used when it is over a certain length in relation to the parallel walls to either side but does not prohibit the use of the wall if desired.

Since there is but a single wall considered to be acting on Line C, tributary width is adjusted in accordance with Section 3.3B3a.

Tributary Widths – Figure 2.30B (2-story portion only)

A to C and C to E	$\frac{24'\text{-}0''}{2} = 12'\text{-}0''$ each
Tributary width	Lines A and E $\frac{24.00}{2} = 12.00'$ each
	Line C 12.0 + 12.0 = 24.0'
	Shortest exterior wall = 4'-0"
	Interior wall = 7'-0" + 11'-4" = 18'-4"
	$\frac{L_e}{L_i} = \frac{4.0}{18.33} = 0.218$
Multiplying Factor	1.39 (Figure 3.2)
	(Multiplying Factor will be used on Calc Form 6 as described in next chapter.)

Differential Oscillation

Because of the differences in height, weight and shear-per-foot in the shear walls, the two sections of split-level houses similar to Model C frequently attempt to oscillate differently when undergoing seismic loads. This can cause the sections of the house to alternately pull apart and pound together. Much damage of this type was observed in the San Fernando earthquake. The split-level tie details in Part IV, Details 7/4 through 13/4, are therefore extremely important. No calculations for these ties are required of the designer.

Alternative Split-Level Construction – see next page.

FIGURE 2.31. **Alternative Rear Elevations – Model C.**

Two alternative methods by which the Model C house might be constructed are illustrated in Figure 2.31. In the upper rear elevation, grade is level across the lot and cripple stud walls or basement walls are projected up to the wood frame first floor of the middle level.

In the lower elevation, grade is just below the floor line of the middle level and the lower level requires retaining walls.

Design calculations are not presented for these cases, but representative examples are shown in later chapters and details required for both types of construction are shown in Part IV.

MODEL D

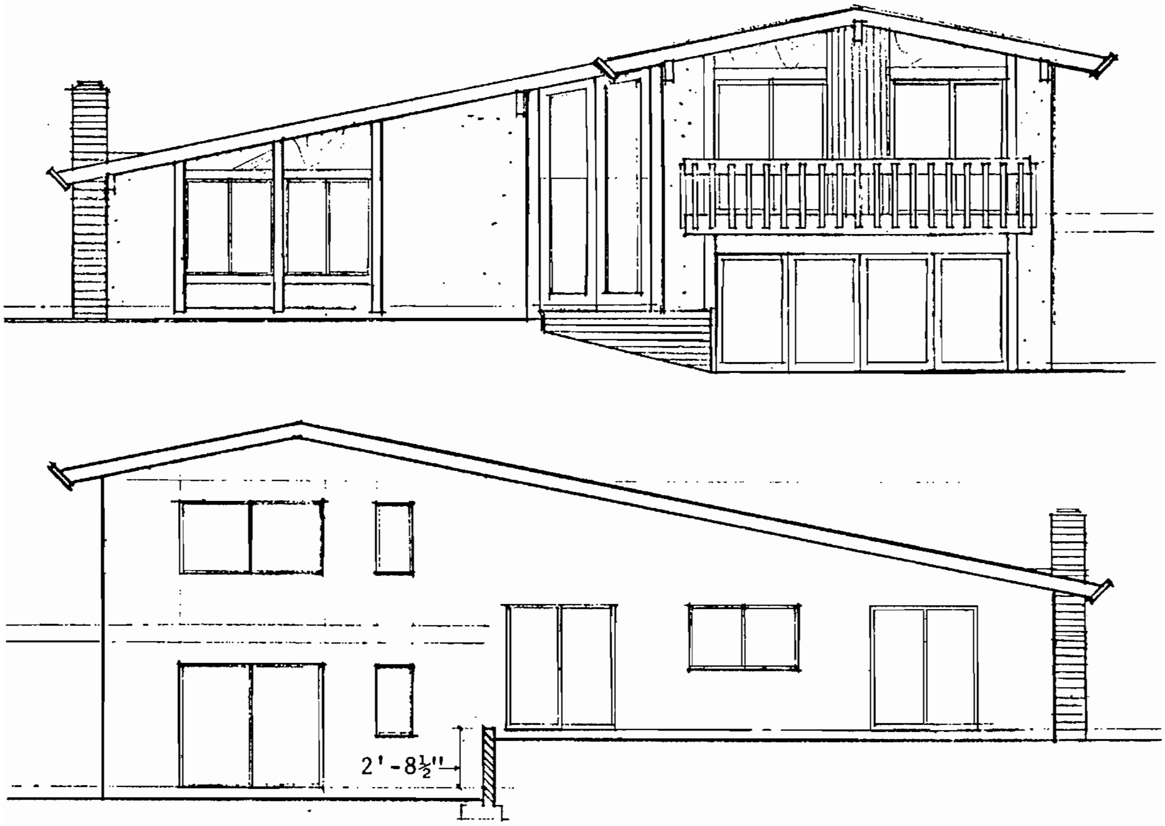


FIGURE 2.32. Front and Rear Elevations – Model D.

This particular model has been made as complicated as the limitations of the report's design methodology allows and it is the only house presented in this chapter that is not similar to any homes encountered in observation of the damage zone in the San Fernando earthquake. Finish materials and framing:

Roof	Roofing	Rock
	Sheathing	2" T & G
	Roof framing	4 x beams
	Ceiling	Exposed
Second Floor	Flooring	Carpet
	Underlayment	5/16" particleboard
	Sheathing	1/2" plywood
	Framing	Wood
	Ceiling	Plaster
Exterior Wall Finish		Stucco
Interior Wall Finish		Gypsum lath and plaster

Plans for the first and second floors and roof of Model D are given in Figures 2.33A and B.

Design Sections in Part III applicable to locating shear walls and determining tributary areas for Model D:

General:	**2.3A1b**	**2.3A2**	
	3.1		
At first story:	**2.2A2**	**2.2B2**	**2.2B3**
	3.3B1	**3.3B3**	
At second story and middle level	**2.1A1**	**2.1A2**	**2.1B**
	2.2A1	**2.2B1**	
	3.3A1		

Model D is similar in floor plan to Model C but it has three unique features: Lack of shear wall at the front of the garage, exposed roof framing and, because of the exposed roof framing, variable height shear walls. These create in one house several special design problems which must be overcome to reduce the vulnerability to damage.

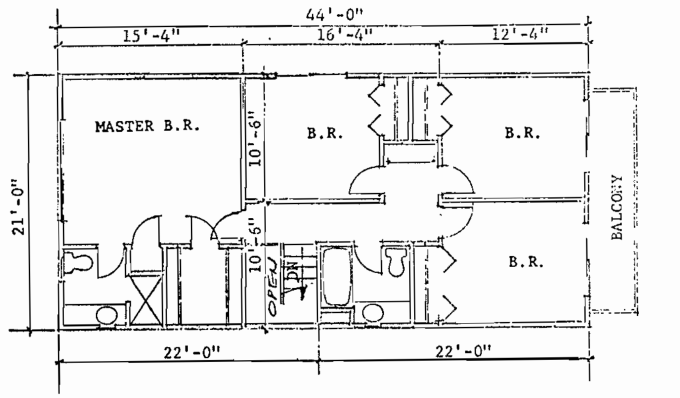
Because of the exposed roof framing, virtually all walls in this residence extend to the roof. The only walls that do not are the entry closet walls at the mid-level and the bathroom and closet walls in the master bedroom at the second-floor level.

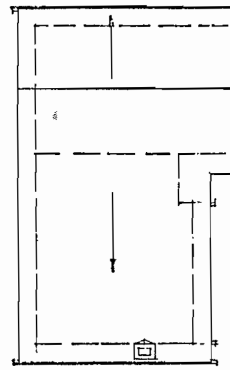
To determine the walls required to be used as shear walls at the second-floor level, Section 2.1B2b, together with its exception, and Section 2.2B2b must be consulted.

Shear wall plans for the first and second floors of Model D are given in Figure 2.34A and B. The tributary area plans for motions in the longitudinal and transverse directions are given in Figures 2.35A and B and Figures 2.36A and B.

Starting at the front interior wall of the second floor the total length of wall at the wardrobes is longer than the exterior wall and therefore must be used. At the rear, the wall between the bathroom and the wardrobe does not extend to the roof and therefore need not be considered. The two walls on Line C are longer than the rear wall and must also be considered as shear wall. The only remaining transverse wall is the wall adjacent to the stairway. The diaphragm ratio in this area is 14.0 / 21 = 0.67, or less than 3/4:1. In accordance with Table 3.1 the wall must be at least 0.85 as long as the shortest adjacent wall to be required to be used. The wall is 7.33 / 13, or 0.56 as long and need not be used. Note that the diaphragm ratio figured in this example uses the width between shear walls on either side of the wall under consideration. The depth is the total depth between the same shear walls.

At the middle level, the shortest length of shear wall along lines A or E is 12′ along Line A. The shear walls occurring in the two-story portion are not considered when the mid-level is being designed. Inspection of the rear elevation (Figure 2.32) will indicate that this assumption leads to some inaccuracies since the whole rear wall obviously acts as a unit. This assumption is made, however, to allow a simple method of analysis as well as for the reasons discussed later comparing Models C and D. The mid-level diaphragm ratio between Lines A and E (Figure 2.35B) is 26/31 = 0.84. The ratio of the wall along Line B to the wall along Line A is 10.33/12 = 0.86. The length of the wall on Line B is therefore greater than the





ROOF PLAN

FIGURE 2.33A. Second Floor Plan – Model D.

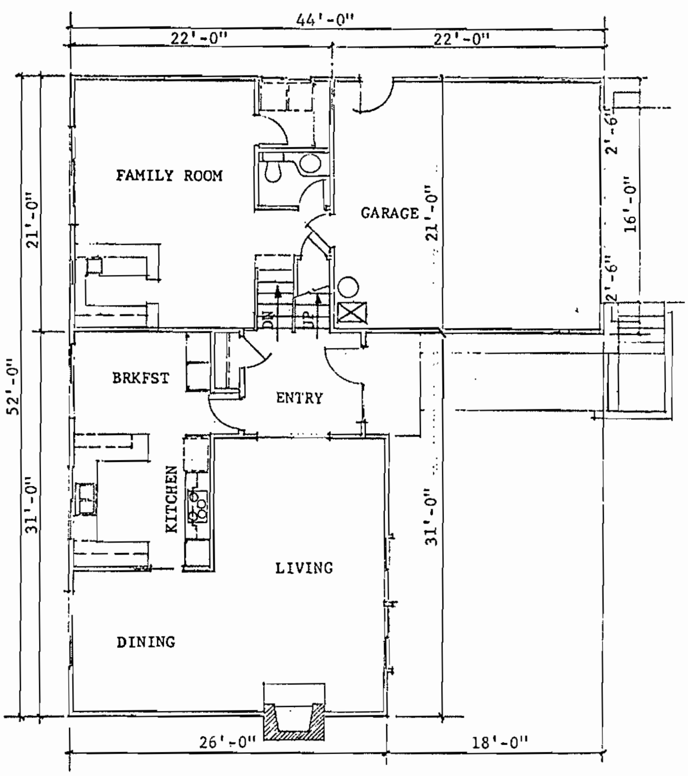


FIGURE 2.33B. First Floor Plan – Model D.

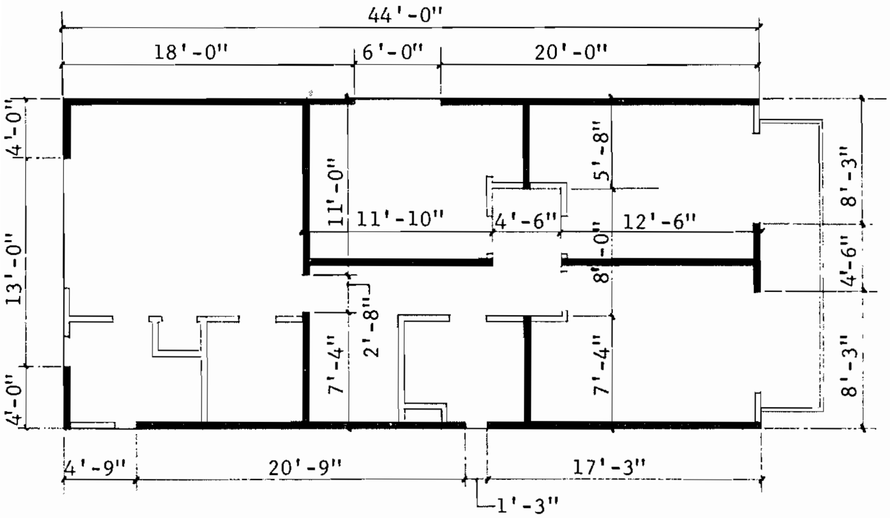


FIGURE 2.34A. Second Floor Shear Wall Plan – Model D.

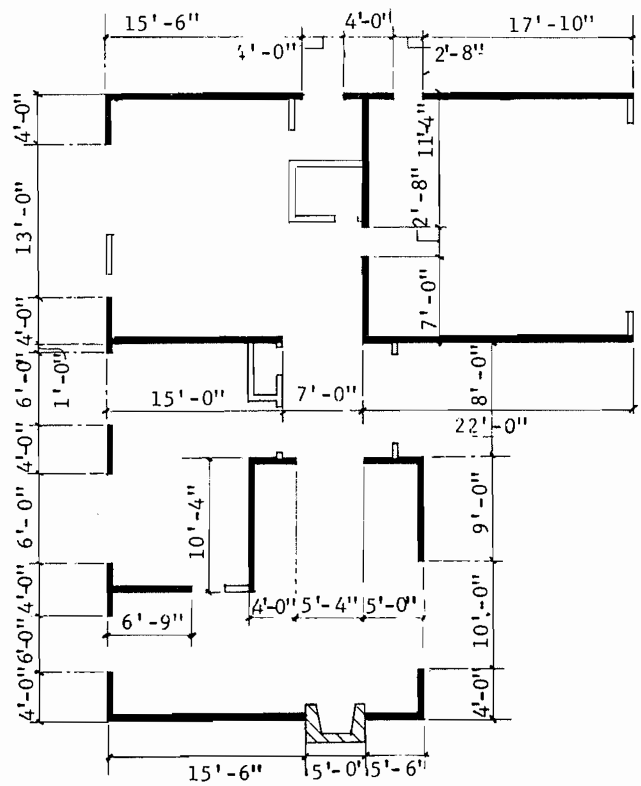


FIGURE 2.34B. First Floor Shear Wall Plan – Model D.

0.80 allowed for diaphragms with ratios not exceeding 1:1 per Table 3.1 and must be used. In the other direction the diaphragm ratio is 31/26 = 1.19. The walls along Lines I and J are obviously not .75 times as long as the walls on H and K and therefore need not be used. They will be used, however, for purposes of this example.

The adjusted tributary width to Wall D is considerably different than the tributary width to Wall C of Model C. Referring to Section 3.3B3a, L_e is defined as the total effective length of the shortest exterior wall. In this example, the shortest exterior wall would be at the front of the garage and the effective length of eligible shear wall at this location is required to be considered as zero. L_e / L_i is therefore also zero and Figure 3.2 requires the multiplying factor to be 2.0. This can in some circumstances result in adjusted tributary widths greater than the width of the house (Section 3.3B4). Since no shear wall is shown on Line G, the design at that location is in violation of Sections 2.2B2a or 2.2B3. This, then, is a special case and is to be handled as discussed below.

Many split-level designs incorporate the garage as shown with Models C and D. Because of the width of the two-story sections on these two models it is possible to incorporate shear wall at the front of the garage. Many homes are not quite as wide and the provision of a two-car garage door and four feet of shear wall is not always possible. When shear wall is not provided at this location it is usually impossible to cantilever the diaphragm in accordance with the Design Methodology and, when no shear wall is provided, this particular configuration is extremely susceptible to high damage and even collapse. If no shear wall is present at the front the diaphragm, in addition to normal deflection, tends to rotate in a horizontal plane about the interior shear walls. Provision of shear wall at the rear of the residence (Line A on Figure 2.35B for instance) does not seem to prevent this rotation and, indeed, field observations indicate that the stiffer the wall at the rear of the house the more likelihood for damage. This same type of rotational effect can be observed to a lesser degree in more normal two-story residences; it is partly for this reason that the multiplying factor is required at all first-floor interior walls.

To avoid precluding a very common type of design, special provisions are set forth in this report to allow this particular configuration in split-level design to continue to be used. The detail indicated for this condition (Detail 46/4) will allow greater deflections than are normally encountered in shear walls. This special detail is intended to prevent rotation of the diaphragm only, however, and no credit is given to it as a shear-resisting element. Some shear will obviously be resisted but the amount in relation to the interior wall depends on the architectural design of the house. Special conditions must also be attached to the use of this detail. It shall be used at garages only and is not meant for use to eliminate shear wall requirements where glazing occurs along the same line. If the front of the mid-level is in line with the front of the garage the strap shown on Detail 13/4 should also be installed. This strap is required any time walls line up, as occurs at the rear of Model D.

Figure 2.35B indicates that if the bathroom wall to the left of Line D is considered a shear wall the tributary width to Line D will be considerably reduced. In accordance with subsection 3.2B this wall, if considered as a shear wall, may, but need not be considered as acting together with the wall on Line D since it is more than 6′ but less than 9′ from Line D. By using this wall, only the width of the garage and half the width to the bathroom wall would be considered tributary to Line D. This would overcome the intent of the first portion of Section 3.3B4 and that is why the second portion of that Section is added, requiring that all interior walls also be considered as acting together and be designed for 100% of

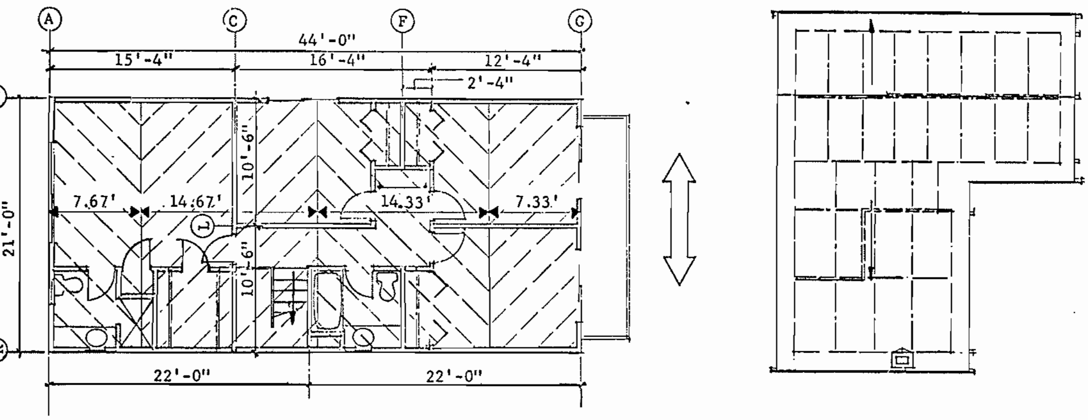


FIGURE 2.35A. Tributary Areas, Transverse Direction — Second Floor, Model D.

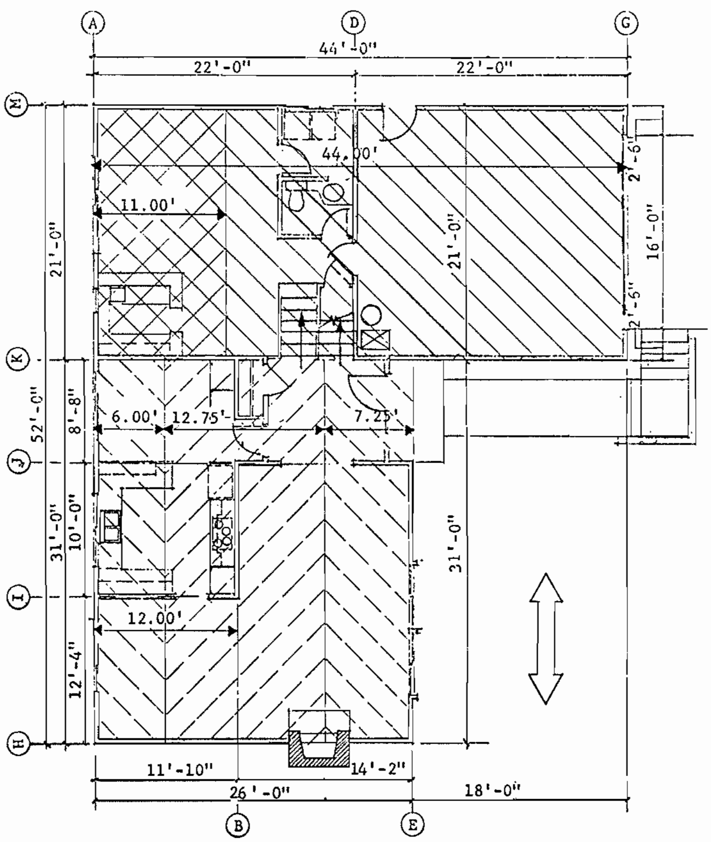


FIGURE 2.35B. Tributary Areas, Transverse Direction — First Floor, Model D.

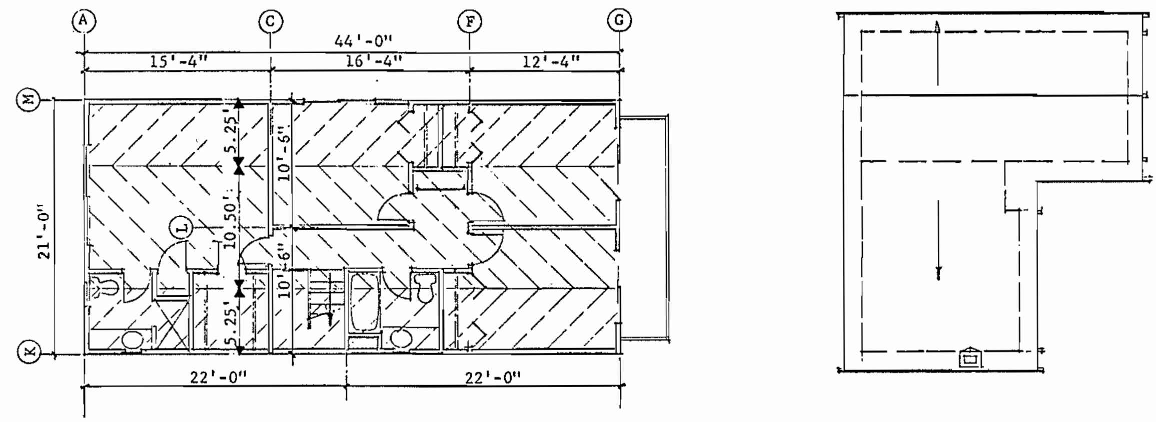


FIGURE 2.36A. Tributary Areas, Longitudinal Direction – Second Floor, Model D.

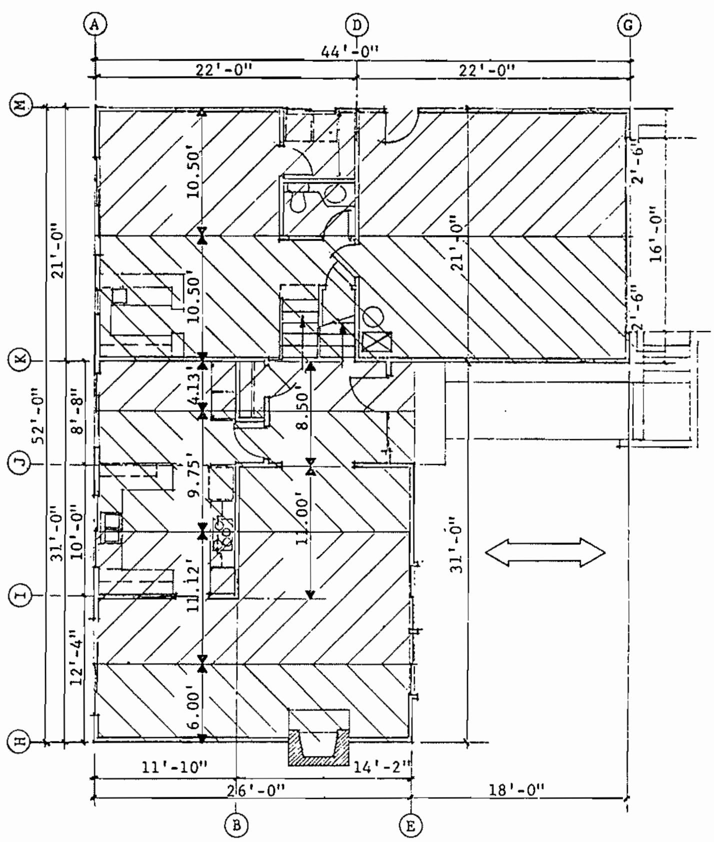


FIGURE 2.36B. Tributary Areas, Longitudinal Direction – First Floor, Model D.

the two-story area as well as the normal method, using for each wall whichever load is greater. In view of these requirements, if the bathroom wall were used it would be best to consider it as acting together with the wall on Line D.

Comparison of Model C and Model D

Models C and D appear to be very similar in floor plan. They could, in fact, be considered as variations of the same plan as is so frequently done in tract development. The middle level plans are virtually identical and, although the two-story portions vary by four feet in length, their interior floor plans are also essentially the same. As mentioned previously, because of three major variations in architectural design, the seismic analysis of Model D varies considerably from that of Model C. This is especially evident in the consideration of tributary areas.

Most split-level houses differ from combination one- and two-story homes in that not only the diaphragms but also the top plates of the walls are at different levels. Where the top plates are continuous, such as at Model D, the walls vary considerably in height and, obviously, the total load in the two-story portion is much greater than that in the one-story portion. For these reasons it is necessary to design the middle level separately from the two-story portion.

SUMMARY OF LOCATING SHEAR WALLS AND DETERMINING TRIBUTARY AREAS

All exterior walls 4'-0" and longer may be considered as shear walls. When the house is framed with a sloping roof and a ceiling below, interior walls extending to the roof level are almost always also included. All walls along a given line are termed a line of shear resistance or line of resistance. Usually each line of resistance is considered to act separately regardless of the total effective length of shear wall along the line. When lines are close together they are combined as if they were the same line. The tributary width to lines of shear resistance is 50% of the distance to the line on either side. Rules governing all the conditions mentioned above are set forth in the Design Sections, 2 and 3.

A number of "special" conditions occur so frequently that they are also covered in the Design Methodology. Some of these are:

One-story houses:

1. Discontinuous roofs.
2. Cathedral ceilings and exposed roof framing.
3. Small wings extending from the main body of the house.
4. Flat-roofed houses or other designs with many interior walls extending to the roof level.
5. Fronts of garages where no wall qualifying as a shear wall exists.

Two-story houses and various configurations combining one and two stories.

1. All the items listed above can also occur in these houses.

2. Interior walls below the second-floor diaphragm

3. Long and narrow second-floor diaphragms.

4. Holes in the second-floor diaphragm.

While the Design Methodology governing the determination of walls to be used as shear walls is intended to leave the designer with considerable latitude, it is nevertheless set forth as a specification. When conditions exist that do not conform to the provisions regarding shear walls, the house should be engineered.

The guidelines for the determination of tributary areas should also be followed in most cases. Conditions can exist, however, requiring judgmental decisions in determining these areas to meet the intent of the provisions (see Appendix B). That is why the Design Methodology subsections concerning tributary areas are termed Guidelines. In all cases, all roof and floor areas should be assigned to a line of resistance.

Observations during the 1971 San Fernando earthquake indicated that greater attention should be paid to certain first-story shear walls in two-story houses. New requirements are set forth for the use of a multiplying factor when first-storv interior shear walls are employed. These requirements were not previously used, even when the house was engineered.

Familiarity with the methods used to figure tributary areas is most important for several reasons. Obviously, the area considered should approximate the actual area contributing load in the event of seismic activity to assure proper shear wall design. In addition, after gaining some experience, the designer or checker will be able to determine "by inspection" that certain lines of shear resistance have more than sufficient shear wall and need not be considered further. This is particularly true of one-story houses. In these instances the tributary area to only one or two lines of resistance need be calculated, eliminating the need for a cross-check of the total area being considered. This type of procedure is not only more efficient, it is also more economical. In adopting it, however, it becomes essential that the "critical" walls checked be figured with a proper understanding of how to determine the areas contributing load to those "critical" walls.

† † † † †

CHAPTER II–3 DETERMINING DEAD LOADS AND SEISMIC LOADS

To design shear walls, the actual loads tributary to the wall must first be determined. Calc Forms 1 through 9 have been supplied to make the task of determining loads and designing shear walls easier. These forms have been used for the determination of loads for Model Homes A, B, C and D, as applicable. It is suggested that the reader briefly familiarize himself with Section 4 of the Design Methodology, where application of Calc Forms 1 through 7 is described, and then use Section 4 as a reference when various examples are presented.

As indicated in Chapter II–1, load is a product of area times weight times a seismic factor. The length and width of each area tributary to each line of resistance has now been determined for each example home. (Calculation of area is developed under discussion of Calc Form 6.)

Equivalent Seismic Weight for a material used in constructing a floor or roof is determined by multiplying the material's weight-per-square-foot by the seismic factor; that is, design inertia force contributed by each square foot of a material is equal to its weight times the seismic factor. This has been done for the designer and is presented in Table 3.2. Adding together the Equivalent Seismic Weights of each material in a floor or roof determines the total inertial force per square foot generated by that assembly. This force-per-square-foot is designated as Unit Load. Thus, area times unit load equals the total seismic load generated by a single sub-assembly (such as a floor) to a given line of resistance.

The weights of materials presented in Table 3.2 are conservative only in the sense that they reflect the heaviest "normal" weight when a given material has a range of weights. They are not intended to make a design more conservative than that which would result if the house were engineered.

Because of the complexities in determining the contribution of walls to seismic loads, a simplified method has been developed. Despite the many variations in size and shape, the ratio of square feet of wall to square feet of floor area is reasonably constant for most residences. Since the walls place load on the roof and floor diaphragms, it follows that an Equivalent Unit Load for walls can be devloped based upon the weight-per-square-foot of the wall assembly. This Equivalent Unit Load can be added to the Unit Load developed by the floor or roof to obtain the Total Unit Load to be used.

Exterior walls frequently have a weight greater than interior walls. Thus it is desirable to separate equivalent unit wall load into these two categories. As mentioned previously, many interior walls tend to support themselves – particularly in one-story construction where the ceiling tends to act as a diaphragm. For this reason, interior wall load to roofs need not be as great as to floors. All these conditons are covered by Tables 3.4 through 3.8.

Since the number of walls and the ratio of interior and exterior wall loads to floor area varies somewhat from house to house, the Unit Wall Loads as presented are somewhat conservative but can be exceeded in houses containing many small rooms. Interior Unit Wall Loads are based on the assumption that there is one square foot of wall for each square foot of floor area in one-story houses as well as in the upper floor of two-story houses, and somewhat less than 2/3 square foot per square foot of floor area at the first floor of two-story houses. Exterior walls are assumed to have 1-1/8 times the total area of the floor. As is customary in seismic design, door and window openings have been ignored in computing these areas. The area of wall as mentioned above assumes walls 8' in height. Total area for the walls is therefore arrived at by multiplying total lineal footage by 8.

Since loads in Seismic Zone 3 are twice as large as in Zone 2 and therefore develop more problems of shear resistance, the examples shown are figured for seismic Zone 3. Zone 2 shear wall design for each of the models has been prepared and is presented at the end of the following chapter. For purposes of comparison it should be borne in mind that the equivalent seismic loads shown in the following examples are double those that will be encountered when using the tables in the Design Methodology for seismic Zone 2.

MODEL A

Normally the designer would be quite familiar with what finish materials he intended to use. In this case these materials have been specified as each model has been introduced in the previous chapter. The listing for Model A reveals that a single finish material is used throughout for each component – the roof, the ceiling, the interior walls and the exterior walls with the exception of the veneer. Calc Form 1 therefore may be used in lieu of Calc Forms 2 and 3 in accordance with Design Section 4.1.

Calc Form 1

The actual and equivalent seismic weights are determined from the appropriate Table 3.2

for seismic Zone 2 or for seismic Zone 3. After listing all weights for the roof, ceiling, exterior walls and interior walls, a total for each group should be obtained. As explained in the following chapter, it is important to obtain the total actual weight as well as the total of the equivalent seismic weights (the Unit Load) for the roof and ceiling. To preclude the necessity of figuring framing weights and at the same time include a small amount of weight for blocking, kickers or truss webs, etc., it is required that the weight of roof and ceiling framing each be assumed to be 2 pounds per-square-foot minimum. Where trusses are used the top chord of the truss is assumed to be roof framing and the bottom chord ceiling framing. It is also required that wall framing be assumed to weigh 4 pounds per-square-foot for reasons similar to those stated above. In addition, miscellaneous weight not figured, such as insulation, cabinets, etc., become a much larger percentage of the weight of walls with relatively lightweight finish materials than is the case with heavier finishes. It is therefore further required that no finished wall be assumed to weigh less than 10 pounds per-square-foot. This minimum weight will be assumed for all future calculations in place of the 8 psf shown on this form.

The veneer which occurs from Line C to Line D along Line E as shown on Figure 2.17 (pg. II–27) is full height and is figured as follows:

Thickness of veneer	= 4 inches
Weight	= 4″ x 10psf = 40psf (Section 4.3C)
Actual Weight	= 40 psf (From Table 3.3)
Equivalent Seismic Weight	= 5.33 psf (Table 3.3)
Equivalent seismic load to roof	= 5.333 x 8/2 x 8/8 = 21.33plf • (Section 4.3C2)

A more complete discussion of figuring veneer loads is presented following the examples in this chapter.

Unit roof load is figured separately from unit ceiling load. Because of roof overhangs, the area of the roof is generally much greater than the floor area of a house. The ceiling, exterior wall and interior wall loads are predicated upon the floor area. Since roof overhangs vary the roof load must be determined using the actual area of the roof itself. Load differentials caused by the slope of the roof are neglected. When roof slopes are steeper than 6 in 12 it is recommended that the roof load be increased by multiplying the actual length of roof framing that occurs in a one-foot horizontal distance by the entire roof load. The equivalent seismic load should be equally adjusted. No form has been provided for this adjustment but it is suggested that the calculation be made directly below the tabulation for the roof on Calc Form 1. For a roof slope of 12 in 12, for instance, the loads would be multiplied by 1.4/1 and the actual load would then be 9.1 psf and the equivalent seismic load would be 1.214.

Since unit ceiling and wall loads will be multiplied by the floor area, they are combined at the bottom of the sheet into a unit load termed "ceiling" load. This load does not represent the seismic loading caused by the ceiling alone but rather the unit load of the exterior and the interior walls as well as the ceiling. The exterior unit wall load is obtained from the appropriate Table 3.6 (Zone 3) considering a wall weight of 10 psf. Interior unit wall load is similarly determined from Table 3.4 (Zone 3), again using 10 psf.

CALC FORM 1

Job EXAMPLE HOMES Model 'A'

ONE-STORY RESIDENCE
ROOF, CEILING AND WALL WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	CEILING	Material	Actual Weight	Seismic Weight
Roofing	WOOD SHAKE	3.0	0.400	Framing		2.0	0.267
Sheathing	SPACED	1.5	0.200	Finish	GYP. BD./ ACOUS. FIN.	3.0	0.400
Framing	TRUSSES	2.0	0.267				
TOTAL		6.5	0.867	TOTAL		5.0	0.667
			Unit Load				Unit Load

EXTERIOR WALLS	Material	Actual Weight	Material	Actual Weight	INTERIOR WALLS	Material	Actual Weight
Framing					Framing		
Finish	PLYWD.				Finish	GYP. BD.	
Finish	GYP. BD.				Finish	" "	
TOTAL					TOTAL		

ROOF UNIT LOAD: 0.867 psf

VENEER LOAD: 21.33 lb/ft

	"CEILING" UNIT LOAD
Ceiling	0.667
Exterior Walls	0.750
Interior Walls	0.250
TOTAL	1.667 psf

Calc Form 6

In determining dimensions of tributary areas, the additions shown in this tabulation are normally done mentally but are shown in this first example for clarity. For all example homes the overhang distance is assumed to be two feet. Note that for Line A, for instance, the area of the floor is 10 x 27.5 = 275 square feet while the roof area is 378 square feet. If the roof overhangs were not considered, more than 100 square feet of roof area would be eliminated from the calculations. Roof Area, then, is the area of the roof over the floor area tributary to any given line of shear resistance; Ceiling Area is the actual floor area as shown on the floor plan in the previous chapter.

The calculations for areas as presented appear imposing (where overhang is not listed separately, calculations would take approximately 1/2 the page) but with experience in this type of design, it will be necessary to check only certain lines calling the rest "ok by inspection." For Model A an experienced designer would check the walls on Lines C, E and F only (see Figure 2.17) and the calculations would take a total of 3 lines. This should be borne in mind when reviewing the remaining Calc Forms for Model A as well as the Calc Forms for the other example homes.

Calc Form 7

The actual seismic load to each line of shear resistance is calculated on Calc Form 7. Roof and ceiling areas are obtained from Calc Form 6 and loads from Calc Form 1. In addition, the fireplace load of 600 pounds required by Table 3.2 is added to Lines B and F (because the fireplace falls within the area assigned to these lines), and the veneer load is added to Line D (for similar reason) as shown under the columns headed Total Roof Load. The total load for each wall is obtained by adding the roof and "ceiling" loads together. Since no ceiling occurs in the garage, ceiling loads for this area have been omitted but exterior wall load is still considered as "ceiling" load. Note that interior wall loads may also be eliminated from garage areas when considereing one-story construction.

CALC FORM 6

Job EXAMPLE HOMES Model 'A'

TRIBUTARY AREAS

	ROOF AREA		CEILING AREA	
LINE	Length x Width	Area	Length x Width	Area
A	(10+2)x(27.5+4)			
	=12x31.5	378	10x27.5	275
B	28.33x31.5	892	HSE 18.33x27.5	504
			GAR. 10x27.5	275
C	(7.5+2)x(18+2)			
	=9.5x20	190	7.5x18	135
D	(18.67+2)x(27.5+2)			
	+9.5x20			
	=20.67x29.5		18.67x27.5	
	+9.5x20	800	+7.5x18	648
		2260		1837
E	(13.75+2)x(57+4)		HSE. 13.75x37	509
	=15.75x61	961	GAR. 13.75x20	275
F	13.75x61+		HSE. 13.75x37	
	(22.58-15.75)x(15+4)		+8.83x15	641
	=961+6.83x19	1091	GAR. 13.75x20	275
G	(9.17+2)x(15+4)			
	=11.17x19	212	9.17x15	138
		2264		1838

CALC FORM 7

Job EXAMPLE HOMES Model 'A'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
A	378	0.867			275	0.750	328 206 534
B F.P.	892 (PER	" TABLE	3.2)	773 600 1373	275 504	0.750 1.667	1373 206 840 2419
C	190	"			135	1.667	165 225 390
D VNR.	800	"	15×21.33=	694 320 1014	648	"	1014 1080 2094
E	961	"			509 275	1.667 0.750	833 848 206 1887
F F.P.	1091 (PER	" TABLE	3.2)	946 600 1546	641 275	1.667 0.750	1546 1068 206 2820
G	212				138	1.667	184 230 414

MODEL B

Calc Form 2

Loads for roof and ceiling are established in the same manner as for Model A. Calc Form 2 must be used because second-floor load must also be determined. Because the floor joists are smaller than 2 x 12s at 16, 4 psf may be used for the second-floor framing weight.

Calc Form 3

Wall loads are less than 10psf and 10psf should be used for all future calculations.

Veneer load is figured in the same manner as for Model A.

Exterior wall loads for the ceiling of the second floor are determined from Table 3.6 and interior wall loads from Table 3.4.

At the second floor, exterior wall loads are doubled while interior wall loads are obtained from Table 3.5. Once again the reader is reminded that these loads are for seismic Zone 3. Note that the total equivalent seismic load that will be used at the second floor line is obtained by adding the second-floor ceiling load to the second floor load.

Since 1/2 the wall weight from above the second floor contributes load to the second floor as well as 1/2 the wall weight below, the exterior wall load at the second floor is double that at the roof. At the interior walls the walls parallel to the direction of load have been omitted from loads at the roof line since these walls usually stop at the ceiling line and carry their own weight down to the ground in one-story construction. The load has also been reduced at the roof line because the interior walls parallel with the load carry some of the cross partition load. In two-story construction, however, all load due to second floor walls must be carried by the second floor diaphragm. For this reason, the load factor obtained from Table 3.5 for second floor load is six times as great as the load to the roof level as obtained from Table 3.4.

Calc Form 6

The calculations on this form are much more extensive than would be the case in actual practice. The walls normally checked would be the first story walls on Lines B, D, E, F and I. Since there is no "ceiling" load associated with the porch roof (the exterior walls are considered part of the house or the garage) and since there is no roof load at the second floor, these columns are left blank.

Calc Form 7

Note that fireplace load is added to the walls at both the high roof and at the second floor. Because Line D at the low roof has several entries two lines are used for figuring the total load with the total on the first line omitted. This procedure serves to warn the designer that the total load occurs farther down the sheet. The same procedure is used for Lines A and B for the first-story walls. The latter walls also receive veneer load as is shown on page II–69.

CALC FORM 2

Job EXAMPLE HOMES Model 'B'

ROOF, CEILING AND FLOOR WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	Material	Actual Weight	Seismic Weight
Roofing	ASPHALT SHINGLE	3.0	0.400			
Sheathing	3/8" PLYWD.	2.0	0.267			
Framing	STD. FRMG.	2.0	0.267			
TOTAL		7.0	0.934			

CEILING						
Framing		2.0	0.267			
Finish	GYP. BD.	2.0	0.267			
TOTAL		4.0	0.534			

2d FLOOR						
Flooring	CARPET	1.0	0.133			
Sheathing	5/8" PLYWD.	2.0	0.267			
Framing	2x10@16	4.0	0.533			
Ceiling	GYP. BD.	2.0	0.267			
TOTAL		9.0	1.200			

CALC FORM 3

Job EXAMPLE HOMES Model 'B'

WALL WEIGHTS – SEISMIC UNIT LOAD SUMMARY
(In Pounds Per Square Foot)

EXTERIOR	Material	Actual Weight	Material	Actual Weight	Material	Actual Weight
Framing		4.0				
Exterior Finish	HDBD.	2.0				
Interior Finish	GYP. BD.	2.0				
TOTAL		8.0				

INTERIOR

Framing		4.0				
Interior Finish	GYP. BD.	2.0				
Interior Finish	" "	2.0				
TOTAL		8.0				

VENEER LOAD: 21.33 lb/ft

TOTAL UNIT LOADS	Roof	2ND CLG.	2ND		GAR.	
Ceiling		0.534	1.200			
Exterior Walls		0.750	1.500		0.750	
Interior Walls		0.250	1.500			
TOTAL	0.934	1.534 +	4.200 =	5.734	0.750	

CALC FORM 6

Job EXAMPLE HOMES Model 'B'

TRIBUTARY AREAS

	ROOF AREA		CEILING AREA	
LINE	Length x Width	Area	Length x Width	Area
HIGH ROOF				
A	20 x 32	640	18 x 28	504
B	20 x 32	640	18 x 28	504
		1280		1008
E	16 x 40	640	14 x 36	504
I	16 x 40	640	14 x 36	504
		1280		1008
GARAGE ROOF				
B	10.17 x 28.67	292	10.17 x 24	244
C	12.5 x 28.67	358	10.5 x 24	252
		650		496
D	14.67 x 22.67	333	12 x 20.67	248
H	14 x 22.67	317	12 x 20.67	248
		650		496
PORCH ROOF				
A	5.67 x 7.67	43		
B	18 x 7.67	138		
		181		
D	5.33 x 23.67	126		
E	2.33 x 23.67	55		
		181		
2ND FLOOR				
A			18 x 28	504
B			18 x 28	504
E			7.0 x 36	252
E			17.93 x 36 (INF)	645
I			6.42 x 36	231

CALC FORM 7

Job EXAMPLE HOMES Model 'B'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
HIGH	ROOF						
A F.P.	640	0.934		598 600 1198	504	1.534	1198 773 1971
B	640	"			504	"	598 773 1371
E	640	"			504	"	1371
I F.P.	640	"		598 600 1198	504	"	1971
LOW	ROOF						
D GAR.	333	"			248	0.750	311 186
PORCH D	126	"					118 615
C	358	"			252	0.750	334 189 523
H	317	"			248	"	296 186 482

CALC FORM 7

Job EXAMPLE HOMES Model 'B'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
2ND FLOOR A PORCH	43	0.934		1198 40 1238	504	5.734	1238 2890
F.P. VNR A			18×2133 =	600 384 984			984 5112
B GAR.	292	"		598 273 871	244	0.750	871 183
PORCH VNR. B	138	"		129 384 513	504	5734	513 2890 4457
E PORCH	55	"		598 51 649	252	"	649 1445 2094
F&G					645	"	3698 3698
I F.P				1198 600 1798	231	"	1798 1325 3123

MODEL C

Calc Form 2

Blank spaces have been left on the form below the normal materials used for each framing group. In Model C underlayment is used and is included in one of these spaces under second floor load.

Calc Form 3

Although wall weight is calculated to be 8psf the minimum 10psf is again used for exterior and interior walls.

No veneer is utilized.

The loads obtained for exterior and interior walls in the bottom table are obtained from the same tables as used for Model B.

Calc Form 6

In the previous examples the areas in each direction have been added together as a cross-check. Due to small arithmetical deviations areas may occasionally vary by one or two square feet as is the case at the mid-level roof. Since the multiplying factor is required for Wall C below the second floor it is not possible to add these areas and obtain equal answers. No additions are therefore shown.

Walls that would normally be checked for Model C are Lines B and D at the mid-level and Lines C and E at the first story of the two-story portion. First-story interior walls in two-story houses should normally be checked. Interior wall loads may neither be eliminated nor reduced for garage areas when the garage is part of two-story construction since interior partition load occurs above the garage.

CALC FORM 2

Job EXAMPLE HOMES Model 'C'

ROOF, CEILING AND FLOOR WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	Material	Actual Weight	Seismic Weight
Roofing	ASPHALT SHINGLE	3.0	0400			
Sheathing	1/2" PLYWD	2.0	0.267			
Framing	TRUSSES	2.0	0.267			
TOTAL		7.0	0934			

CEILING						
Framing		2.0	0.267			
Finish	GYP. BD.	2.0	0.267			
TOTAL		4.0	0534			

2d FLOOR						
Flooring	CARPET	1.0	0.133			
Sheathing	1/2" PLYWD	2.0	0.267			
Framing	2x14@16	5.0	0.667			
Ceiling	GYP BD.	2.0	0.267			
UNDER-LAYMENT	5/16" PTCLEBD	1.5	0.200			
TOTAL		11.5	1.524			

CALC FORM 3

Job EXAMPLE HOMES Model 'C'

WALL WEIGHTS – SEISMIC UNIT LOAD SUMMARY
(In Pounds Per Square Foot)

EXTERIOR	Material	Actual Weight	Material	Actual Weight	Material	Actual Weight
Framing		4.0				
Exterior Finish	FIBERBD.	4.0				
Interior Finish	GYP. BD.	2.0				
TOTAL		10.0				

INTERIOR						
Framing		4.0				
Interior Finish	GYP BD	2.0				
Interior Finish	" "	2.0				
TOTAL		8.0				

VENEER LOAD: ____________ lb/ft

TOTAL UNIT LOADS	BOTH Roofs	MID & 2ND CEILING	2ND			
Ceiling		0.534	1.524			
Exterior Walls		0.750	1.500			
Interior Walls		0.250	1.500			
TOTAL	0.934	1.534 +	4.524 =	6.058		

CALC FORM 6

Job EXAMPLE HOMES　　　　Model 'C'

TRIBUTARY AREAS

LINE	ROOF AREA Length x Width	ROOF AREA Area	CEILING AREA Length x Width	CEILING AREA Area
MID-LEVEL ROOF				
B	15x33	495	13x31	403
D	15x33	495	13x31	403
		990		806
F	17.75x30	533	15.75x26	410
G	15.25x30	458	15.25x26	397
		991		807
2 STORY ROOF				
A	26x25	650	24x21	504
E	26x25	650	24x21	504
		1300		1008
G	12.5x52	650	10.5x48	504
H	12.5x52	650	10.5x48	504
		1300		1008
2ND FLOOR				
A			12x21	252
C			24x21x1.39	701
E			12x21	252
G			10.5x48	504
H			10.5x48	504

CALC FORM 7

Job EXAMPLE HOMES Model 'C'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
MID--	LEVEL						
B	495	0.934			403	1.534	462 618 1080
D F.P.	495	"		462 600 1062	403	"	1062 618 1680
F F.P.	533	"		498 600 1098	410	"	1098 629 1727
2ND STORY ROOF							
A	650	"			504	"	607 773 1380
E	650	"			504	"	1380
G	650	"			504	"	1380
H	650	"			504	"	1380

CALC FORM 7

Job EXAMPLE HOMES   Model 'C'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
2ND FLOOR A					252	6.058	607 1527 2134
C					701	"	4247 4247
E					252	"	607 1527 2134
G					504	"	607 3053
MID-LEVEL G	458	0934			397	1.534	428 609 4697
H					504	6.058	607 3053 3660

MODEL D

Model D is much more complicated than the other examples and it might be advisable to obtain engineering for houses of this type. The recommended provisions in this report can provide a design for this type of house but great care would be required. Some of the complications become apparent in the discussion in the next chapter on shear wall design.

Since the Calc Form information otherwise is similar to that in the previous example, only Calc Form 5 and Calc Form 7 are commented upon in detail.

Because of the use of plaster, exterior and interior wall loads are 20psf on Calc Form 3.

Calc Form 5

At the mid-level it is necessary to adjust the exterior and interior wall heights. Since the walls extend to the ceiling, loads for the interior walls are obtained from Table 3.7 and the exterior wall loads from Table 3.6. The average height of both categories, exterior and interior, is assumed to be 12.5 feet. It is assumed throughout the calculations that the walls extend to the underside of the sheathing.

The second-story walls of the two-story portion must also be adjusted. Calc Form 5 is used rather than Calc Form 4 because the walls are full height and do not stop at the ceiling. Interior wall load to the roof is obtained from Table 3.7 and to the second floor from Table 3.8. Note that Table 3.8 does not list the entire load to the floor, only that portion developed from below the second floor. The total load to the second floor is therefore obtained by adding the value shown in Table 3.8 to the value shown in Table 3.7 as indicated in Note 1 of Table 3.8.

Calc Form 7

The roof load developed in a given wall is normally considered to remain in that wall. In the case of Lines D, K and M, however, shear walls exist above them which extend from the roof to the second floor but then stop at that point. It is therefore necessary to add the loads from Lines A, C, F and G to obtain the total roof load tributary to Line D – all load is considered to be tributary to this line as shown on Figure 2.35B. In the longitudinal direction, as shown on Figure 2.36A, Line L falls exactly midway between K and M. One-half the load in Wall L is therefore added to Line M and one-half to Line K.

CALC FORM 2

Job EXAMPLE HOMES Model "D"

ROOF, CEILING AND FLOOR WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	Material	Actual Weight	Seismic Weight
Roofing	ROCK	6.0	0.800			
Sheathing	2" T&G	4.0	0.533			
Framing	4x BEAMS	2.0	0.267			
TOTAL		12.0	1.600			

CEILING						
Framing						
Finish						
TOTAL						

2d FLOOR						
Flooring	CARPET	1.0	0.133			
Sheathing	1/2" PLYWD.	2.0	0.267			
Framing	2x14@16	5.0	0.667			
Ceiling	PLASTER	8.0	1.067			
UNDER-LAYMENT	5/16" PTCLEBD	1.5	0.200			
TOTAL		17.5	2.334			

CALC FORM 3

Job EXAMPLE HOMES Model 'D'

WALL WEIGHTS – SEISMIC UNIT LOAD SUMMARY
(In Pounds Per Square Foot)

EXTERIOR	Material	Actual Weight	Material	Actual Weight	Material	Actual Weight
Framing		4.0				
Exterior Finish	STUCCO	8.0				
Interior Finish	LATH & PLASTER	8.0				
TOTAL		20.0				

INTERIOR

Framing		4.0				
Interior Finish	LATH & PLASTER	8.0				
Interior Finish	"	8.0				
TOTAL		20.0				

VENEER LOAD: ________ lb/ft

TOTAL UNIT LOADS	BOTH Roofs	MID LEVEL CEILING	2ND STY. CLG.	2ND FLOOR		
Ceiling		—	—	2.334		
Exterior Walls		2.213	1500	3.000		
Interior Walls		1.966	1583	2.423		
TOTAL	1.600	4.179	3.083 +	7.757 =	10.840	

CALC FORM 5

Job EXAMPLE HOMES Model 'D'

UNIT WALL LOADS ADJUSTED FOR HEIGHT
Houses With Flat Roofs or Sloping Ceilings

	Adjustment	= W (psf)	Adjustment	= W (psf)	Adjustment	= W (psf)
ROOF						
$W_R = \frac{w_r h_r}{8}$	$\frac{1.50 \times 11.8}{8}$	= 2.213	$\frac{1.333 \times 11.8}{8}$	= 1.966	$\frac{\quad \times \quad}{8}$	=
FULL HEIGHT INTERIOR WALLS						
$W_2 = \frac{w_r h_r}{8}$	$\frac{1.333 \times 9.5}{8}$	= 1.583	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
$+ \frac{w_2 h_2}{8}$	$\frac{\quad \times \quad}{8}$	= 0.840	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
$= W_2$		2.423				
EXTERIOR WALLS						
$E_r = \frac{e_r h_r}{8}$	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
$E_2 = E_r + \frac{e_2 h_2}{8}$	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
$= E_2$						

CALC FORM 6

Job EXAMPLE HOMES Model 'D'

TRIBUTARY AREAS

	ROOF AREA		CEILING AREA	
LINE	Length x Width	Area	Length x Width	Area
LOW	ROOF			
A	792 x 33	261	5.92 x 31	184
B	13.0 x 33	429	13.0 x 31	403
E	9.08 x 33	300	7.08 x 31	219
		990		806
H	8.42 x 30	253	6 42 x 26	167
I	11.42 x 30	343	11.42 x 26	297
J	908 x 30	272	9.08 x 26	236
K	4 08 x 30	122	4.08 x 26	106
		990		806
2ND STORY ROOF				
A	9 67 x 25	242	7.67 x 21	161
C	14.67 x 25	367	14.67 x 21	308
E	14.33 x 25	358	14.33 x 21	301
G	9.33 x 25	233	7.33 x 21	154
		1200		924
K	7.25 x 48	348	5.25 x 44	231
L	10.50 x 48	504	10.5 x 44	462
M	7.25 x 48	348	5.25 x 44	231
		1200		924
2ND FLOOR				
A			11 x 21	231
D			22 x 21 x 15	924
K			10.5 x 44	462
M			10.5 x 44	462

CALC FORM 7

Job EXAMPLE HOMES Model 'D'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
MID-LEVEL A	261	1.600			184	4.179	418 769 1187
B F.P.	429	"	$\frac{600}{2}$ =	686 300 986	403	"	986 1684 2670
E F.P.	300	"	$\frac{600}{2}$ =	480 300 780	219	"	780 915 1695
H F.P.	253	"		405 600 1005	167	"	1005 698 1703
I	343	"			297	"	549 1241 1790
J	272	"			236	"	435 986 1421

CALC FORM 7

Job EXAMPLE HOMES Model 'D'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
2ND STORY ROOF							
A	242	1.600			161	3.083	387 496 883
C	367	"			308	"	587 950 1537
F	358	"			301	"	573 928 1501
G	233	"			154	"	373 475 848
K	348	"			231	"	557 712 1269
L	504	"			462	"	806 1424 2230
M	348	"			231	"	557 712 1269

CALC FORM 7

Job EXAMPLE HOMES Model 'D'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
2ND F A	LOOR				231	10.840	387 2504 2891
D	A C E G	387 587 573 373 1920			924	"	1920 10016 11936
M	M ½ L	557 403 960			462	"	960 5008 5968
K	K ½ L	557 403 960			462	"	960 5008
MID-LEVEL K	122	1.600			106	4.179	198 443 6609

VENEER LOADS

Veneer load is not included with other wall loads and is figured on a per-foot basis with all load falling along the edge of any tributary area being added into the load of that area. Each wall should therefore be figured separately.

Veneer is assumed to carry its own weight when the wall is parallel to the direction of seismic motion. In addition, when one-inch-thick grout reinforced with paper-backed 2" x 2" x 16 gage galvanized wire mesh is used for tied veneer, or cement plaster reinforced with either paper-backed metal lath or wire mesh as described above is used for adhered veneer, the shear value for stucco may be assumed for the outside face of the wall.

If veneer thickness (including thickness of backing) is 4" the weight per-square-foot is 4" x 10psf = 40psf. For seismic Zone 3 equivalent seismic weight is 5.333psf per Table 3.3.

In accordance with Section 4.3C of Part III the following examples indicate the method of figuring the load per foot imposed on the roof or second-floor diaphragms.

One Story – Wall Height = 8'-0"

Veneer height = 2'-0" Neglect, no calculation necessary

Veneer height = 3'-0" Load to roof $\frac{5.33 \times 3}{2} \times \frac{3}{8} =$ 3.00 lb/ft

Veneer height = 4'-0" Load to roof $\frac{5.33 \times 4}{2} \times \frac{4}{8} =$ 5.33 lb/ft

Veneer height = 6'-0" Load to roof $\frac{5.33 \times 6}{2} \times \frac{6}{8} =$ 12.00 lb/ft

Veneer height = 8'-0" Load to roof $\frac{5.33 \times 8}{2} \times \frac{8}{8} =$ 21.33 lb/ft

Two Story – Wall Height = 8'-0"
Framing Height = 1'-0"

Veneer to 2nd floor line

Load from below 2nd	=	21.33
Load at 2nd	=	5.33
		26.66 lb/ft

Veneer to 2'-0" above 2nd

Load to roof $\frac{5.33 \times 2}{2} \times \frac{2}{8} =$ 1.33

Load to 2nd $2 \times 5.33 - 1.33 =$ 9.33

Load below 2nd	21.33
Load at 2nd	5.33
Load above 2nd	9.33
	35.99 lb/ft (36)

Two Story — Wall Height = 8'-0" (Continued)
Framing Height = 1'-0"

Veneer to 4'-0" above 2nd	Load to roof (see one-story)	5.33 lb/ft
	Total weight above 2nd (4 x 5.33)	21.33 lb/ft
	Load below 2nd	21.33
	Load at 2nd	5.33
	Load above 2nd (21.33–5.33)	16.00
		42.66 lb/ft
Front and back both veneered (example above = 2 x 42.66		85.33 lb/ft

WIND LOADS

In many parts of seismic Zone 3 local building departments require that wind load be checked as well as seismic load. Section 601–6 of the Manual of Acceptable Practices discusses wind loads extensively. This information will not be repeated here, and this report does not discuss the design of individual elements (rafters, studs, etc.) of a residential structure for wind load. The load to individual shear walls can be governed by wind, however, such that shear-per-foot, sill bolts, etc., are frequently governed by wind rather than seismic load. Recommended wind load tables from the MAP are reproduced in this report and should be compared with the requirements of local building departments to determine the maximum wind load design required in a particular area. Since the MAP is recommended only, no requirements are set forth for wind load other than those contained in the Minimum Property Standards for One- and Two-Family Dwellings.

The comments with regard to wind load are therefore confined to those requirements that may be in conflict with the recommended seismic provisions and it should not be interpreted that this report sets forth all requirements necessary for wind load design. Roof uplift, for instance, can be a major factor in high wind areas and in open-walled construction. Modes of failure, in fact, are quite different for wind loads than for seismic loads. The effect on shear walls is quite similar, however. Figure 2.2 compares wind and seismic forces. As implied in the figure, wind load is simpler to figure in that the total horizontal projected area of the building is the area subjected to wind load. Since the load from the bottom half of the first-floor walls is transmitted directly to the ground, the wind load on this area may be neglected. Also usually neglected are the roof overhangs at each end of the elevation of the house being considered. Referring to Figure 2.14, for instance, the height of the stud walls is 8' and the height to the peak of the roof above the top plates of the wall is 4'-11" at a 4 in 12 slope. Total height affecting the shear walls would therefore be 1/2 the wall height plus the roof height equals 4'-0" + 4'-11' = 8'-11."

Tributary widths to individual shear walls or lines of resistance are figured in the same manner as they are for seismic loads with the exception that the multiplying factor need not be applied to interior walls. The wind load to Line A on Figure 2.18 would therefore be 8.92 x 10.00 x wind load per-square-foot = 89.2 x wind load. If a wind load of 15 lb per-square foot is assumed, the total load to Line A would be 8.92 x 10.0 x 15 = 1338 lb. Calc Form 9 provides columns for calculating wind load and is discussed in the following chapter.

SNOW LOADS

When snow exists on a roof at the time of seismic disturbance, some of its weight will contribute to the horizontal force generated in the structure. Current practice uses from 25% to 100% of the required snow load when calculating the seismic loads. This portion of the load is then considered as part of the dead load when determining the seismic load. In view of recent considerations with regard to this subject, in areas requiring vertical load designs for snow loads of 32psf or less, the snow may be ignored when figuring unit loads. In locales where vertical load design requires snow loads of greater than 32psf the equivalent seismic weight of 25% of the snow load shall be considered in determining the unit load of the roof.

Because of varying practices, the designer should be careful to determine local requirements in order to verify that they are not more severe than those mentioned above.

In entering the snow load on Calc Forms 1 and 2 "Snow Load" should be entered under the other roof load designations in one of the two blank spaces provided under the word "Framing." The equivalent seismic weight should be obtained from Table 3.3.

† † † † †

CHAPTER II–4 WOOD FRAME SHEAR WALL DESIGN

Use of Calc Form 8 (or 9)

Once loads to each line of resistance are established design of individual shear-resisting elements is the final step in the design of the overall structure for seismic forces. Calc Form 8 is used for this purpose. Calc Form 9 may be substituted if wind load design is also desired.

The three items that must be checked in designing each shear wall are:

1. Capacity of the shear-resisting material contemplated vs. the actual load-per-foot in the shear wall.
2. Sill bolt spacing such that the shear in the wall can be transferred to the foundation or basement walls.
3. Stability of the wall. If the wall proves to be unstable, hold-down anchors must be used to resist overturning.

Capacity of Shear-Resisting Materials

Table 3.9 lists various shear-resisting materials together with the allowable shear-per-foot for each material and the connections required to obtain this allowable shear. This table is repeated as Table 4.3 and covers all broad classifications of materials (other than plywood) currently accepted for lateral load shear resistance. In accordance with MPS requirements, other materials or methods of attachment must be approved by ICBO and HUD acceptance procedures prior to their use. Some "brand name" products already have such approvals. Before using such products, the designer is advised to request a copy of the ICBO approval from the manufacturer's representative. This approval will state the allowable shear-per-foot and construction requirements for the product as well as indicate that the product is, indeed, approved by ICBO.

Table 3.10 indicates acceptable thicknesses for plywood sheathing and lists the allowable shear value for each combination of thickness and nailing. Other methods of fastening are approved by HUD-FHA for the fastening of plywood. These are contained in HUD-FHA bulletin number UM-25d. Although many methods of attachment are listed, the designer is cautioned to determine that the method proposed is acceptable to local building officials. Generally speaking, staples having the same allowable shear value as the nailing required are acceptable to most building departments.

Tables 3.12 through 3.15 graphically illustrate the total allowable load a shear wall can carry depending upon its length and the type of material applied.

To determine the adequacy of a given shear-resisting material the designer may either divide the total load along a given line of resistance by the total length of shear wall available to obtain the shear-per-foot, or he may consult Tables 3.12 through 3.15 and obtain the approximate shear-per-foot directly.

Sill Bolt Spacing

Figure 3.16 graphically illustrates the allowable load for gun bolts and 1/2" and 5/8" round anchor bolts. In addition to the graphic illustration the values for up to 4 bolts of each type are tablulated. In short walls it is usually simpler and better to determine the total load in a given length of wall and call out the actual number of bolts required using the tabulations shown. Since 3/4" round bolts are often required in short segments of wall the values for these bolts are also shown in the tabulation.

Gun bolts (power driven studs) shall be used for interior walls only and should be the size specified in Section 6.2D of the Design Methodology. Maximum spacing of gun bolts is 3'-0" and minimum spacing is 6" o.c.

Whenever possible it is suggested the designer call for the same size anchor bolts throughout the residence as fewer errors are likely to occur when this is done. Since minimum bolts are 1/2" round at 6' o.c., an attempt should be made to use 1/2" round anchor bolts wherever possible. Spacing should never be less than 12" o.c. however. Anchor bolts should extend through finish floor slabs on grade and into the foundation itself a minimum of 4".

On many occasions a window or other wall opening occurs adjacent to the shear wall such that wall finish material 2'-0" high or greater occurs below the opening. In this case the total length of sill plate may be considered in determining the shear-per-foot used to determine sill bolt spacing. In such instances, the shear-per-foot for sill bolt spacing will not be the same as the shear-per-foot in the shear wall itself. Since all finish materials should be fastened as shown in Table 3.9 or 3.10 the finish material below the opening will be capable of transferring the load along the wall to the points where the bolts occur. As an example, the side wall of a house near a property line might be 30 feet long with 15 feet of windows and 15 feet of shear wall. If the windows have normal sill heights and no doors are present the length of wall for determining shear-per-foot would be 15 feet but the length of wall for determining sill bolt spacing would be 30 feet.

Overturning

Recommended overturning provisions are for determination of those walls that are obviously unstable and the size hold-down anchors required to correct the condition.

Effective Length of Wall

As stated in Design Section 2.1A1 the effective length of a shear wall is the length of wall extending uninterrupted from the sill plate to the diaphragm. If that length of wall were all that were considered for overturning many more walls than necessary would require hold-down anchors.

Frequently an opening, such as a window, is adjacent to the shear wall. For overturning purposes, the window may often be considered as an opening in a longer wall rather than as a complete interruption of the shear wall. If, for instance, a ten-foot-long shear wall contains a one-foot-square hole at its exact center, it may be visualized that the wall will act as a unit and that the hole will have little effect on the wall for overturning purposes. For shear, however, the one foot would have to be subtracted from the ten feet of length and it would be considered that the effective length of the wall was 9'-0". If the one-foot-square opening is gradually enlarged in both directions, the point will come when the remaining wall above and below the opening is too limber to allow the two sections of wall on either side to be considered as a unit for purposes of overturning. In the nominal 8'-high wall the maximum height of opening allowed is 4'-3", provided the opening is so located that a minimum of 2'-6" of wall height occurs either above or below the opening. When openings higher than this occur in 8'-high walls the wall segments above and below the opening are not considered strong enough to assist an adjacent shear wall in resisting overturning.

The strength of the wall at the opening is also affected by the length of the wall segments. For this reason the width of opening is restricted to 2.5 times the height of the highest portion of wall above or below the opening but in no case shall an opening be wider that 8'-0". For 2'-6"-high segments the maximum width of opening is therefore restricted to 6'-3" and for sections 3'-3" and higher the width is restricted to 8'-0". Where wider openings occur with no intermediate support for the header above, the wall above and below the opening should not be considered as contributing to the resistance to overturning of the adjacent shear wall.

It is not necessary to have shear wall on each side of an opening to increase the effective length of a wall for overturning considerations. A post, for example, is sufficient at the far side of the opening to consider the length as extended, provided the shear-resisting material is properly fastened to the post. To consider the wall extended even farther, however, a full-height wall as wide as the height of the highest portion of wall above or below the opening (but it need not exceed 4'-0") must be provided on the opposite side of the opening from the shear wall.

Figure 2.39 illustrates several examples of walls that can be considered as "solid" walls for purposes of overturning. The term "solid" wall is used to define a wall whose length for overturning purposes may be increased to the length it would be if the wall were actually solid with no opening. In the upper one-story example in Figure 2.39 the section of wall at the far left must be considered as acting by itself since the door opening adjacent is more than 4'-3" in height. The center section of wall containing the 5'-wide opening may be considered to be 12'-4" in length when considering overturning since the section of wall below the window is 2'-6" high and only twice that in length. Note that even if the section below the windows were 3'-3" in height, thereby qualifying the 8'-0"-wide window to the right, the effective length of this segment of wall would remain the same. The portion between the windows is only 1'-4" in width, which is less than the height of the portion below the windows (2'-6") and the wall cannot be extended farther. Since the 8'-0"-wide opening does not qualify, the section ofwall on the right hand end must be considered as acting alone. Again, if the section of wall below the window were 3'-3" the right hand wall could then be considered as being 24' in length and the 1'-4" section between the windows would be considered as a part of each section of wall even though the two walls could not be considered as one.

In the center example the left hand portion may be considered as being 16' long since the opening does not exceed 8' in length. The portion over the sliding glass door opening may not be considered since too little wall occurs over the opening. The two 4'-long shear panels must be considered as separate elements since the opening between them is 10' in length. If a post were placed at the center of the 10'-0" opening each of the 4' panels could be considered 9'-0" long but again could not be considered as extending farther.

For wall heights other than 8'-0" the height of the opening may not exceed 55% of the height of the wall and a minimum of 2'-6" of wall must occur either above or below the opening. Width requirements for openings are the same as stated above. When the shear material is discontinuous at the second floor framing the same rules as set forth above apply to each floor of two-story houses. When the finish material is continuous past the second floor framing (as usually occurs with stucco) the height to the sill of second-floor openings may be considered the height of the first floor wall. This principle is demonstrated in the bottom example of Figure 2.39. At the left end the height of the wall becomes 8'-0" plus 1'-0" for framing plus 3'-3" to the sills for a total height of 12'-3". Maximum height of opening is therefore 12.25 x .55 = 6.74'. This, for all intents and purposes, is 6'-9" and the wall above the 8'-wide opening can then be considered as extending the shear wall for overturning purposes for a total length of 21'-0" at the left hand side. Near the right hand end the second floor window has a lower sill and the opening itself is 10'-0" in width. For each of these reasons the right-hand segment of wall must stop at the edge of the 10'-wide opening. The 5'-6"-high opening is meant to represent framing around a fireplace. The wall above the opening at this point is high enough to allow the segment adjacent to the 4'-long shear wall to be considered as acting with the wall for overturning purposes.

Vertical Load to Wall

To use the overturning tables in Design Section 6 it is necessary to know the vertical load imposed on the wall by roof and floor framing as well as the weight of the wall itself. When rafters, ceiling joists or floor joists are supported on the wall the load from these elements is equal to 1/2 their span (plus any cantilever) times the vertical load as developed on Calc Form 1 or 2. This principle is discussed further in the commentary on Models A and B. Wall weight per-square-foot should be considered to be the same as was used in computing lateral load (10psf minimum) but weight of veneer should be excluded from the vertical load calculations.

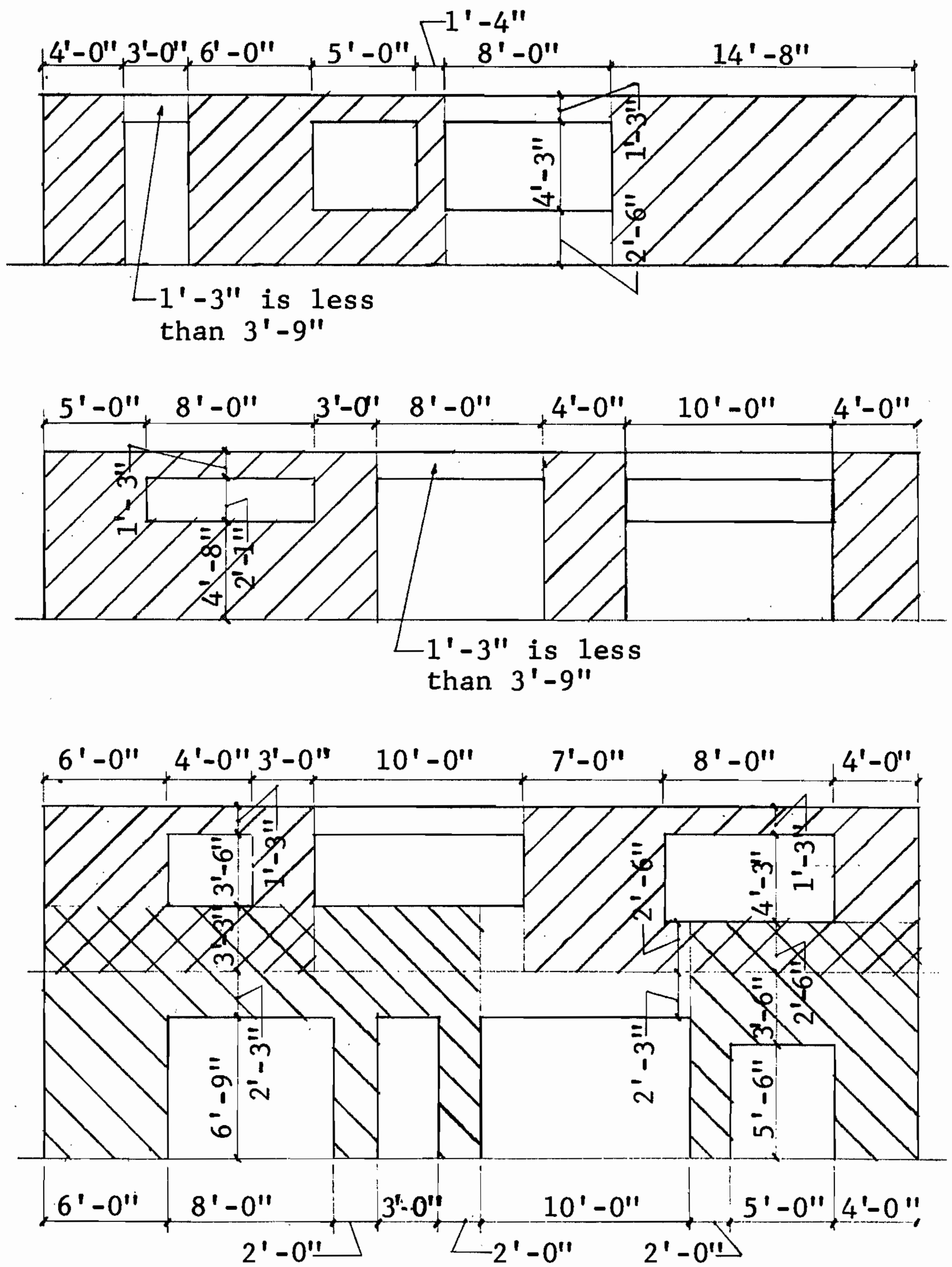


FIGURE 2.39 Determination of Effective Length of Walls for Overturning.

Step-by-Step Determination of Hold-Down Requirements

A step-by-step approach is set forth here to illustrate the determination of whether hold-down anchors are required for overturning resistance. The simplified methods indicated, while reasonably accurate, are somewhat liberal for short, non-bearing walls and conservative for longer bearing walls. Since short walls are usually the walls requiring hold-down anchors in residences, it is suggested that the designer be no more liberal than the method indicates. Overturning stability of walls shall be determined as follows:

1. **Determine effective length of wall for overturning.** The methods for doing this were discussed above.

2. **Determine vertical load-per-foot.** The weight of the wall including superimposed dead load of the roof, ceiling and second floor must be determined.

3. **Determine horizontal load capacity for each length of wall along a given line of shear resistance.** This can be accomplished by consulting the graphs in Design Section 6 entitled "No Hold-Down Anchors Required." These graphs are presented for vertical loads of 60 pounds-per-foot and 100- through 600-pounds-per-foot in 100-pound increments. Allowable horizontal load for other vertical loads may be determined by interpolating between graphs. Each graph has the portion of the graph that is hardest to read enlarged on a graph on the facing page. No reinforcing steel is required in the footings when these graphs prove the wall to be sufficient. A method for dealing with walls having a vertical load greater than 600 pounds-per-foot is shown in Appendix A.

   It is generally advisable when performing this step to start with the longest effective length of wall since very often one wall will be capable of resisting the entire horizontal force to a given line of resistance. If this proves to be the case, no further checking is required. Since this method is approximate, a tolerance of 5% is acceptable in making this and subsequent determinations; i.e., actual load may exceed allowable load by 5%.

4. **Add together the allowable horizontal force for each segment of wall along a given line of resistance.** Because headers framing into the ends of walls and walls at right angles to the shear wall provide additional vertical load resistance to overturning, an arbitrary additional horizontal force of 100 pounds may be added for each length of wall along the line of resistance. A length of wall is considered to be the length used in determining overturning resistance and is not to be confused with the number of shear walls present. The sum of the allowable horizontal forces, plus the 100-pound force times the number of wall lengths, may prove to be greater than the horizontal load along the line of resistance. If this is true, no further design is required.

5. **When the horizontal force is greater than the resistance offered with no hold-down anchors apply the formula:**

$$U = \frac{H}{L} \times P_{act} - P_{all})$$

where

H	=	Height of wall
L	=	Total lenght of all "solid" walls and single shear walls along the line of resistance. Example: Top of Figure 2.39 where L = 4'-0" + 12'-4" + 14'-8" = 31'-0".
P_{act}	=	Actual horizontal force along the line of resistance
P_{all}	=	Allowable horizontal force as determined from Step 4 above.
U	=	Uplift load generated

Figure 23/4 indicates the allowable force for a BO framing anchor as being 400 pounds upward. Detail 26/4 indicates how these framing anchors may be installed to resist upward forces. When one framing anchor is used the allowable uplift force is 400 pounds. Two such framing anchors may be used to resist forces of up to 800 pounds. The typical strap hold-down anchor (Detail 25/4) is capable of resisting 1700 pounds.

If U is less than 1700 pounds one of those three methods of hold-down may be used. The hold-downs indicated must be installed to a double stud (or larger) at each end of each "solid" wall segment along the line of resistance and should be called for on the foundation plan. When the strap hold-down is used a #4 bar should be placed top and bottom in the footing. The bars should extend the length of the wall plus 6'-0" beyond at each end.

6. **When U is greater than 1700 pounds the graphs for hold-downs Numbers 1, 2, 3 and 4 should be consulted** consecutively until the sum of the horizontal forces from each length of wall is greater than the actual horizontal force. This hold-down must then be used at each end of each wall length. The formula given in Step 5 is not applicable to these heavier hold-downs.

7. **When using hold-downs 1, 2, 3 and 4 a reinforced grade beam must be utilized** in conjunction with the hold-down anchor. The size and reinforcing of the grade beam required is indicated on the hold-down graph. Details are presented in Part IV – Details 28/4 to 34/4. In conjunction with seismic design the term grade beam refers to a reinforced continuous footing, often larger than the other continuous footings, which is used to resist the overturning of shear walls. The only bending developed in this type of grade beam is caused by lateral loads. For vertical load, the beam acts as a continuous footing resting on natural grade.

As shown on Details 30/4 and 31/4 the grade beam need not extend continuously in-line with the shear wall but may turn around corners when the shear wall is located at or near the end of the footing. In all cases the "a" dimension of the reinforcing steel must be observed. In the case of interior walls whose footings would normally stop at the end of the wall the grade beam should be extended a sufficient distance to develop the "a" distance of the reinforcing steel. When such an interior wall footing intersects a perpendicular continuous footing extending to either side of it the reinforcing should be extended in both directions as shown on Detail 31/4 such that the total length of reinforced grade beam beyond the end of the shear wall is equal to "a." Although extending the reinforcing in footings perpendicular to the shear wall is acceptable it is better to have the grade beam extend continuously outward at each end as shown on Detail 29/4. When the footing is in a straight line, in plan view, it provides the most effective resistance to overturning.

Because of frost conditions, basements, etc., the footing might often be deeper than the minimum depth indicated on the grade beam details. When the beam is deeper, less reinforcing may be used. The area of reinforcing utilized can be decreased proportionately from the ratio of the depth, as shown on the grade beam detail, to the increased depth from the top of the wall to the reinforcing. For example, if a 4'-0"-deep footing is used in a condition requiring a 12 x 18 grade beam with two #6s top and bottom, the depth to the center of the reinforcing indicated on the grade beam detail is 14-5/8", while in the actual condition the depth would be 44-5/8". The area of steel in the 14-5/8"-deep grade beam is 0.88 square inches. The area of steel required in the 4'-deep footing would be 14.625/44.625 x .88 = 0.288 square inches. Two #4s or one #5 could be substituted for the two #6s called for when this depth of footing is used. In the case of 8'-0"-high basement walls no reinforcing is required for any of the hold-downs when the wall is of concrete. For masonry walls of the same height, two #4s or one #5 top and bottom may be used with any hold-down.

MODEL A

General

Calc Form 9 and the vertical wall load calculations provide the design for the shear walls for Model A in Zone 3. Calculations for all models in Zone 2 are appended at the end of the chapter. Since the same principles apply, very little comment will be made with regard to the Zone 2 calculations.

When using either Calc Form 8 or 9 it is usually best to complete the design for one wall before proceeding to the next. This will ensure that ample space has been left for the design of each wall. It is usually advisable before proceeding to list the value of the shear material for the first wall to be designed directly under the words Shear Material in the column heading. Since most areas require wind load design, columns for the determination of these loads have been incorporated into Calc Form 9.

Vertical Load

Before designing the shear walls for overturning it is necessary to obtain vertical load tributary to each wall. The calculations on Page II–95 accomplish this. Since each wall has different types of load tributary to it no Calc Form is provided for this purpose. Note that veneer loads are not included in wall weight when figuring the vertical loads available for resisting overturning.

The load to Lines A, B, C and D should suffice as examples of how to obtain the vertical loads. On Line A the trusses are assumed to be at 2′-0″ o.c. with a 2′-0″ overhang. The 2′-0″ overhang plus one-half the distance to the first interior truss is therefore assumed to be the width of the area contributing vertical load to the wall – in this case 3′-0″. The vertical roof and ceiling loads are shown on Calc Form 1; therefore, 3 x 6.5 is equal to the roof load tributary to the wall.

Since 10 psf was used as the wall weight in obtaining the seismic load, the same weight may be used in figuring the vertical load. Wall A has a gable roof and is therefore higher than 8′-0″. It is acceptable to use 8′-0″ as the height when figuring overturning for walls under gable roofs, but this lesser height must then also be used in figuring wall weight.

At Line B there is a truss each side of the line and therefore the tributary roof width is 2′-0″. Since ceiling occurs on one side only, the tributary width of the ceiling is 1′-0″. Wall weight is obtained as described above. For walls under gable roofs it should be noted that the 8′-0″ height assumption is applicable only to walls having continuous top plates extending between the exterior cross-walls.

On Lines C and D the trusses span from Line C to Line D, a distance of 15′. The roof load tributary to each line is therefore 15.0/2 = 7.5′ + 2′ of roof overhang for a total of 9.5′. Ceiling tributary load is obtained in the same manner, except that there is no overhang. The same calculations are made for each wall in the house so that the designer will know which overturning table to consult to determine whether hold-down anchors are required.

EXAMPLE HOMES – MODEL 'A' – VERTICAL LOADS/FOOT

LINE A

W = ROOF – 3.0 x 6.5 = 19.5
WALL – 8.0 x 10.0 = 80.0
99.5 – SAY 100#/1

LINE B

W = ROOF – 2.0 x 6.5 = 13
CLG – 1.0 x 5 = 5
WALL – 8.0 x 10.0 = 80
98#/1 – SAY 100#/1

LINE C AND D – F TO G

W = ROOF – 9.5 x 6.5 = 62
CLG – 7.5 x 5.0 = 38
WALL – 8.0 x 10.0 = 80
180#/1

LINE D – E TO F AND G – C TO D

W = ROOF – 3.0 x 6.5 = 19.5
CLG – 1.0 x 5.0 = 5.0
WALL – 8.0 x 10.0 = 80.0
104.5#/1

LINES E & F – B TO D

W = ROOF – 15.75 x 6.5 = 102
CLG – 13.75 x 5.0 = 69
WALL – 8.0 x 10.0 = 80
251#/1

LINE F – A TO B

W = ROOF – 102
WALL – 80
182#/1

Wall A

The second column of Calc Form 9 indicates the tributary height for wind load. This is normally the total height of the residence minus one-half the height of the first-story wall. The width listed in the third column is the same as the tributary width for seismic loads except that no multiplying factor is used for interior walls. The required wind loading should be determined from the wind zone tables given in the MAP and duplicated in this report as Tables 3.10 and 3.11, or as required by the local building department, whichever value is higher. A wind load of 15psf is assumed in these examples. Total wind load is obtained by multiplying the figures in columns 2, 3 and 4. Thus for shear wall A the wind load is 8.92 x 10.0 x 15 = 1,338 pounds.

Total seismic load is entered in the column with that heading and is obtained from Calc Form 7.

Total effective length of shear wall along the line of resistance is entered in the column headed Wall Length.

Shear-per-foot is obtained by dividing total wind load or total seismic load (whichever is greater) by effective wall length. In this case the calculation is 1,338/27.5 = 48.7 pounds-per-foot. Figure 3.16 indicates that 1/2" round sill bolts at 6' o.c. are good for approximately 145 pounds-per-foot. This is greater than the shear in the walls and no further checking of this item is required.

Since the shear-per-foot in Wall A is less than the 160 pounds-per-foot allowable for the shear material with minimum nailing, the material may be noted as acceptable.

Wall A is one solid wall 27.5 feet long with a vertical load of 100 pounds-per-foot. By consulting the No Hold-Down Required graph for 100 pounds-per-foot vertical load it can be seen that this wall is capable of sustaining a horizontal load of 4,720 pounds which is greater than the 534-pound total seismic load. Note that the examples do not check the shear walls for overturning caused by wind load. In areas where wind load must be checked for this condition the No Hold-Down Required graphs are still applicable.

Wall B

Wind load is obtained in the same manner as for Wall A.

The 2'-8" door opening is subtracted from the 27'-6" of wall length to obtain the effective shear wall length of 24'-10".

The shear of 152.7 pounds-per-foot is greater than that which can be sustained by 1/2" sill bolts at 6' o.c. Figure 3.16 must again be consulted to obtain the required spacing of 5'-9" o.c. Since the only opening in the wall is a door, there is no extension of length possible in figuring sill bolt spacing.

In checking for overturning, the 17'-4" segment of wall should be checked first, since this segment might possibly be capable of supporting the entire horizontal load for overturning. In this case the wall has a horizontal load capacity of 1,860 pounds, not in itself sufficient. Notations are therefore made for the 7'-6" wall and, in addition, the 100-pounds-per-wall length is added for each of the two wall lengths. These loads are added together to obtain the allowable horizontal load of 2,410 pounds. This load is only 9 pounds or 0.4% less than the actual load so the walls are acceptable with no hold-down anchors.

CALC FORM 9

Job EXAMPLE HOMES Model 'A'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 1:00	Overturning
A	8.92	10.0	15	1338	534	27.5	48.7	½@6	OK	L=27.5' P_{ALL}=4720
B	8.92	28.33	"	3791	2419	24.83	152.7	½@5.9	OK	OK
	w=	100#/'				17-4	186.0			
						7-6	35.0			
						2×10.0	20.0			
							241.0			
C	5.58	7.50	15	628	390	4.0	156.9	2-½"φ	OK	1BO Erkg Anch.
	w=	180#/'	P_{ALL}=180	+100=280,			$\frac{8}{4}$(390	-280)=220#		
D	8.92	18.67	15	2498	2094	30.5	81.9	½@6	OK	L=45.5' OK
E	6.62	13.75	15	1365	1887	13.5	139.8	½@6	OK	L=15' P_{ALL}=2800#

CALC FORM 9

Job EXAMPLE HOMES Model 'A'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material	Overturning
F	6.62	13.75	15	1365						
	6.83	8.83	"	905						
				2270	2820	25.17	112.0	1/2@6	OK	OK
		L =	12'	900 +	$\frac{82}{100}$	(1800	-900)	=1638		
			8'-10"	485 +	"	(985	-485)	= 895		
			4'-6"	125 +	"	(250	-125)	= 228		
								2761		
						3 x 100	=	300		
								3061		
G	6.83	9.17	15	939	414	9.0	104.4	1/2@6	OK	L=15' P_{ALL}=1400

Wall C

This wall is just 4'-0" long. The actual number of sill bolts is therefore noted, rather than their spacing. Since the vertical load to the wall is 180 pounds-per-foot, when checking overturning it is necessary to interpolate between the graphs for 100 and 200 pounds-per-foot vertical loads. In the case of 4'-0"-wide walls the interpolation is simple since the graphs reveal the allowable horizontal load is equal to the vertical load-per-foot. The total allowable horizontal load for the wall is therefore 180 pounds. Adding the arbitrary 100 pounds per wall, a total horizontal force allowable of 280 pounds is obtained. Since this is less than the 390-pound seismic load, the formula $U = H/L\ (P_{act} - P_{all})$ is applicable. The calculation shown indicates an uplift force of 220 pounds. This is less than 400 pounds and 1 BO framing anchor is therefore acceptable when installed as shown on Detail 26/4.

Walls D and E

These walls are figured in the same manner as was Wall A.

At Wall D the assumption has been made that the windows have sufficient wall above and below them to qualify the wall as "solid." Wall length between the windows of the Master Bedroom and the center bedroom is obviously long enough to consider the wall as "solid" and able to be continued farther. Although no elevation is shown for this wall a sill height of 2'-8" is assumed at the window in the center bedroom. The wall between this bedroom and the bath therefore also qualifies and the wall may be considered one continuous length.

At Wall E the length of wall between C and D as well as the length at the kitchen can be considered "solid." The wall bétween C and D proves to be sufficient for overturning resistance.

Wall F

This wall has two different vertical loads, depending on whether the wall is in the garage or in the house. Since most of the wall occurs in the garage area the lighter vertical load for the garage area has been used throughout. The vertical load of 182 pounds-per-foot requires an interpolation between the 100- and 200-pound graphs. This is figured by entering the load shown by the 100-pound graph and then adding 82/100 of the difference of the load between the allowable loads shown on the 200-pound graph and the 100-pound graph. To the figures for each of the three walls is added 100 pounds for each wall length to attain a total allowable horizontal load of 3,061 pounds.

Wall G

This wall may also be considered "solid." Where the shear-per-foot is less than 145 pounds sill shear need not be further checked since minimum sill bolting is obviously all that is required.

MODEL B

Vertical Load

For two-story dwellings the vertical load must be figured for the second floor and the first floor separately. These calculations are shown on Pages II–101 and II–102.

First Floor Walls

The high roof loads on Lines E and I are figured somewhat differently from the method shown for Model A because standard wood framing is used rather than prefabricated wood trusses. Referring to Figure 2.24A, standard framing would require that the interior wall midway between Lines E and I be used as a bearing wall. The load to Lines E and I, therefore, is equal to one-half the distance from the exterior to the interior bearing wall plus the 2′ overhang for a total of 9′. Standard framing often uses "kickers," extending from the interior bearing wall at the ceiling line up to the roof, in order to shorten rafter span. When this is done more load is taken by the interior wall and less by the exterior wall. In this case the assumption has been made that no kickers are being used.

If this house were framed with prefabricated wood roof trusses it would be desirable to consider the vertical load from the roof distributed in exactly the same manner as is shown for the standard framing in this example. For a wall to overturn, one end must lift up. In this house the wall directly above would also lift up and press against the roof trusses. During an earthquake, therefore, the load would be distributed partially to the interior wall. Use can be made of this for obtaining vertical load resistance to overturning. If the load is assumed to be acting on the interior wall, however, it may not also be used for the exterior walls. The vertical load is therefore best distributed as if the framing were standard. Note that it is necessary to have a wall above or nearly above the first story shear wall in order to make use of this type of consideration. It is also necessary that the top plates of the second floor wall touch or nearly touch the bottom chord of the trusses.

At the garage the assumption is made that the rafters span from the front to the rear wall with rafter ties installed such that no beam is required at the ridge. When a ridge beam is provided, the load to the wall would be developed from a total of 8′ of roof width rather than the 14′ shown.

As can be seen on Calc Form 2 for Model B the second floor load is 9.0 psf. In determining the load to walls parallel to the floor framing, this load is used. For Lines E, F, G and I, however, the vertical load of the interior walls is added to the second floor load to determine the total vertical load to the wall. In this instance 10psf was used as the weight of the walls. Since one square foot of wall is assumed to exist for each square foot of floor area, the 10psf may also be used as the vertical load of the interior partitions on the second floor. The load to Lines E, F, G and I is therefore 9 + 10 = 19psf vertical load.

Second Floor Walls.

These walls are figured in the same manner as the exterior walls on Model A. Since none of the windows is large, the walls may be considered as "solid." Their lengths are therefore such that they prove to be stable.

Garage Walls.

These walls are obviously sufficient with the exception of the wall adjacent to the gar-

EXAMPLE HOMES - MODEL 'B'. VERTICAL LOADS/FOOT

HIGH ROOF

LINES A & B

W = ROOF - 3.0 x 7.0 = 21
CLG - 1.0 x 4.0 = 4
WALL - 8.0 x 10.0 = 80
105 #/1

LINES E & I

W = ROOF - 9.0 x 7.0 = 63
CLG - 7.0 x 4.0 = 28
WALL - = 80
171 #/1

GARAGE ROOF

LINES D & H

W = ROOF - 14 x 7.0 = 98
WALL - = 80
178 #/1

LINE C

W = ROOF - 3.0 x 7.0 = 21
WALL - = 80
101 #/1

2ND FLOOR

LINE A

W = ROOF - = 21
CLG - = 4
2ND FLR - .67 x 9.0 = 6
WALL - 17 x 10.0 = 170
201 #/1

EXAMPLE HOMES - MODEL 'B' - VERTICAL LOADS/FOOT

2ND FLOOR

LINE B

W = LINE A - = 201
GAR RF - 1.0 x 7.0 = 7
208 #/1

LINES E & I

W = ROOF - 9 x 7 = 63
CLG - 7 x 4 = 28
2ND FLR - 7 x (9+10) = 133
WALL - 17 x 10 = 170
394 #/1

LINES F & G

W = ROOF - 14 x 7 = 98
CLG - 14 x 4 = 56
2ND FLR - 14(9+10) = 266
WALL - 8 x 10 = 80
500 #/1

age door. Wall D has a 4'-0"-wide shear panel for which hardboard with shiplap joints does not provide sufficient shear resistance. Since the wall is only 4'-0" long, hardboard with butt joints could be used for this one portion, or plywood could be placed on the interior of the wall. Overturning is figured in exactly the same manner as it was for Wall C in Model A. The uplift load of 674 pounds requires the use of two BO framing anchors. The one-story Special Garage Front Wall Detail (45/4) could also be used at this location if 2'-6" of wall were provided each side of the door opening.

First Floor Wall A

This wall may be considered "solid" and presents no problem in either shear or overturning.

First Floor Wall B

This wall is considerably shorter than Wall A and is required to support not only the house load but half the garage as well. The shear of 331.4 pounds-per-foot requires the installation of 3/8" Structural II plywood. Although Structural II is the notation used throughout these examples, Design Section 6.3C3a stipulates that this designation is merely representative of any acceptably graded plywood other than Structural I or Siding. Siding plywood would also qualify if it were nailed with common nails rather than the casing nails stipulated at the bottom of Table 3.10.

The high shear also requires considerably closer sill bolt spacing. This 18'-6"-long wall is stable for a horizontal load of 4,520 pounds, which is greater than the 4,457 pound total seismic load.

First Floor Wall E

The wall at the bathroom contains two "holes." The medicine chest would be placed between studs and does not interrupt the exterior finish. It may therefore be ignored. Window sizes should be figured as the clear distance between rough framing. On this basis the maximum hole size of 2'-0"-high by 3'-0"-wide (Design Section 2.1A3) is exceeded by the bathroom window and the shear wall must be considered as stopping at the edge of this opening when considering the length of the wall for shear resistance.

The windows in Wall E require the wall to be considered as divided into two lengths for overturning, one each side of the entry. Even though the short piece of wall between the entry doors and the bathroom window is not long enough to be considered shear wall, it can be considered for purposes of overturning. This length of wall is then 18'-0". Since the vertical load is 394 pounds-per-foot and since the 400 pounds-per-foot graph for No Hold-Downs Required indicates an allowable horizontal load well in excess of the actual load, the allowable load for 394 pounds need not be determined exactly. The allowable load is therefore notated as 8100– pounds.

First Floor Walls F and G

The first step in designing these walls is to indicate the allowable shear for the gypsum wallboard each side, with minimum nailing, per Table 3.9. Since the same material is used each side the value shown in the Table may be doubled.

In considering wind load note that the load to these walls is figured for the total height of

CALC FORM 9

Job EXAMPLE HOMES Model 'B'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 330	Overturning
HIGH	ROOF									
A	90	18.0	15	2430	1971	24.0	101.3	—	OK	L=28' OK
B	90	18.0	"	2430	1371	24.0	101.3	—	OK	L=28' OK
E	6.67	14.0	"	1400	1371	18.5	74.1	—	OK	L=36' OK
I	6.67	14.0	"	1400	1971	20.0	98.6	—	OK	L=36' OK
LOW	ROOF								230	
C	8.33	10.5	"	1312	523	18.0	72.9	½@6	OK	L=18' P_{ALL}=2020
D	6.33	12	"	1140	615	40	28.5	2-½"φ	HD BD BUTT	2 BO FRMG ANCHS
	178+	100=	278	U=$\frac{8}{4}$(615-278) =674#						
H	6.33	12	15	1140	482	20.67	55.2	½@6	OK	L=20.67' OK

CALC FORM 9

Job EXAMPLE HOMES   Model 'B'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 230	Overturning
1st STORY WALLS										
A	18.0	18.0	15	4860	5112	24.0	213.0	1/2@4.0	OK	L=28' P_{ALL}=9750
B	18.0	18.0	"	4860						
	8.33	10.17	"	1271						
				6131	4457	18.5	331.4	1/2@2.6	3/8" SI II 8d@4,12	OK
	L=	18'6"	P_{ALL}=	4250	+0.08x	(6375	-4250)	+100 =	4520#	
									180	
E	14.5	7.08	15	1540	2094	18.0	116.3	1/2@6	OK	L=18' P_{ALL}=8100#
F&G	16.75	14.42	"	3623	3698	19.0	194.6	1/2@4.6	OK	L=15' P_{ALL}=6950#
I	14.42	6.50	"	1406	3123	9.0	347.0	2-1/2" Φ EA. PANEL	3/8" SI II 8d@4,12	OK
	L=	10'-0"	-2500							
		4'-6"	-500							
		2x100=	200							
			3200							

the structure even though the shear wall extends only to the second floor. The tributary widths to Walls E and I have been reduced from those considered at the second floor since Walls F and G, acting together, are assumed to take the center half of the structure for wind. This is somewhat different from the method of figuring seismic loads, wherein the exterior walls only are used to take the roof seismic load. The shear of 194.6 pounds-per-foot is greater than allowable for minimum sill bolting and 1/2" sill bolts are required at 4'-6" o.c. in accordance with Figure 3.16. The 15'-long wall proves to be sufficient to resist all overturning load and therefore no further checking is required.

First Floor Wall I

This wall serves as an illustration of the possible economies in design that can be achieved by providing longer lengths of shear wall than are shown on this example. This line has a total of 9'-0" of shear wall, which represents 25% of the total length of the two-story structure along its rear elevation. If the fireplace were shifted around the corner and the 10'-0"-wide sliding doors in the living room were shifted slightly, more wall would occur at the corner and the wall between the family and the living rooms could also be utilized. A total of 18' of shear wall would then be present along this rear elevation rather than the 9' provided. Moving the fireplace could quite possibly cause a similar problem at Line A but if the fireplace were left where it is, a small window could be put in the end wall and the slider in the rear wall reduced to 8'-0". This would increase the length of shear wall to more than 14' along the rear and shear-per-foot would be significantly reduced.

As the design is shown the shear-per-foot in the two rear walls is 347.0 pounds-per-foot. It is to be hoped that combinations of shear-resisting materials will be allowed in the future, and that this shear can then be resisted with the hardboard and gypsum board finishes provided. As the design stands, it is necessary to provide 3/8" Structural II plywood beneath the finish in order to meet the shear requirement. If the shear wall length were increased to 13'-6" the hardboard finish would be sufficient and the plywood could be eliminated.

Note that if the graph portion of Figure 3.16 is used, the indication would be that 1/2" sill bolts would be required at 3'-9" o.c. A notation such as this has relatively little meaning in a short shear wall. It is better to use the tabulation that indicates that two 1/2" sill bolts in each wall will take the load.

The wall height from the head of the sliding door to the sill of the two large windows is 4'-9" and the overall height to the sill is 11'-6". The solid wall above the window is therefore approximately 41% of the total height which is insufficient to allow the wall above the window to be considered as fixing the wall near Line B. On the other hand, the wall near Line A has an opening that extends only to the height of the mantle of the fireplace. This wall can therefore be considered 10'-long for purposes of overturning.

As another example of economy, the combination of the 10'-0" and the 4'-6" walls is just barely stable for overturning. If the shear material were not designed to extend past the second floor framing each wall could only be considered 4'-6" long and an HD #1 would be required at each end of each wall, thereby increasing cost further. It is recognized that functional design considerations must prevail but an understanding of the effect of shear wall length can frequently assist the designer in achieving economies.

MODEL C

Vertical Load

In figuring the vertical load for the mid-level and the upper story of the two-story portion only brief comments are required because the loads are established in virtually the same manner as for Model A. The only exception is that the portion along Line G, which is common to the mid-level, has a small additional portion of roof and ceiling load contributed by the mid-level.

At the first story, the load along Line E is figured at the shear wall; thus the wall height is the full 17'-0" rather than the smaller portion of wall over the garage door. The portion of wall along Line G common to the mid-level has its wall height figured from the top of the retaining wall which extends downward from the mid-level floor. This wall can be considered a part of the footing. Thus, the portion of wall adjacent to the mid-level is only 4'-0" high to the bottom of the second floor. At Lines G and H the 10 psf partition load is again added to the 11.5 pound dead load of the second floor to obtain the 21.5 pounds shown in the calculation.

Mid-Level Walls

The shear-per foot in Walls B, D and F are all relatively low with the result that the sill bolts and shear material are minimal. At Line B the wall below the kitchen window is sufficient to consider the two shear walls on either side to be acting as one length of overturning purposes. This line of resistance is therefore stable. At the front, however, the wall below the window is not high enough to allow the same consideration and the walls when checked prove to have somewhat less capacity than the actual load. Although the actual load is less than 3% greater, one BO anchor has been designed at each end of each wall.

High Roof Walls

The Shear-per-foot at Lines A, G and H is quite low and the shear material presents no problem along these lines. At Line G it is necessary to assume that the interior gypsum board is taking the load since no fiberboard would be installed in the attic area of the mid-level. At Line E the shear is in excess of that allowed for fiberboard and 3/8" plywood must be used. If it is assumed that the window on Line E is merely a window and does not enter out onto the balcony no hold-down problems occur at any of the walls.

At Line H the wall must be assumed to be 20'-4" long. When the shear material is continuous past the second floor the depth of the shear material would be measured from the head of the window below to the sill of the second floor window. This would justify the 8'-0"-long window as acceptable and allow the wall to be considered a "solid" wall 48'-0" long. Since fiberboard is used the material in most cases would not continuously extend past the second floor framing, with the result that unless this window were a high sill the 8'-0"-wide opening would not qualify.

If a sliding door were installed on Line E at the balcony the remaining wall would be 14'-0" in length with an allowable horizontal load of 1,270 pounds. When the arbitrary additional 100-pound horizontal load is added to this the load is approximately the same as

EXAMPLE HOMES - MODEL 'C' - VERTICAL LOADS/FOOT

MID-LEVEL

LINES B & D

W = ROOF - 15x7 = 105
CLG - 13x4 = 52
WALL - 8x10 = 80
237 #/'

LINE F

W = ROOF - 3x7 = 21
CLG - 1x4 = 4
WALL - 8x10 = 80
105 #/'

2ND STORY

LINES A & E

W = 105 #/'

LINE G @ MID-LEVEL

W = ROOF - 12.5x7 = 88
- 1x7 = 7
CLG - 10.5x4 = 42
- 1x4 = 4
WALL - 8x10 = 80
221 #/'

LINE H & EXT PORTION OF LINE G

W = ROOF - 88
CLG - 42
WALL - 80
210 #/'

EXAMPLE HOMES - MODEL 'C' - VERTICAL LOADS/FOOT

1ST STORY

LINES A & E

W = ROOF - 3x7 = 21
CLG - 1x4 = 4
2ND FLR - 1x11.5 = 6
WALL - 17x10 = 170
201#/1

LINE C

W = 2ND FLOOR - 1.33x11.5 = 15
WALL - 8x10 = 80
95#/1

LINE G @ MID-LEVEL

W = ROOF - 12.5x7 = 88
- 1x7 = 7
CLG - 10.5x4 = 42
- 1x4 = 4
2ND FLR - 10.5x21.5 = 226
WALL - 13x10 = 130
497#/1

LINE H & EXT. PORTIONS OF LINE G

W = ROOF - 88
CLG - 42
2ND FLR - 226
WALL - 170
526#/1

the actual seismic load and the wall would be considered acceptable. When hold-downs are required at second-floor walls (no such condition is shown in these examples), Details 27/4 and 35/4 through 38/4 should be reviewed and the detail matching the type hold-down required should be used.

First Floor Wall A

Although it has not been mentioned, many walls have higher wind loads than seismic load. Wall A is in this category. For seismic load, fiberboard sheathing would be acceptable. The shear-per-foot for wind load, however, is 236.9 pounds-per-foot, which exceeds the allowable shear for fiberboard. Based on this shear, sill bolting is required at 3'-9" o.c. and 3/8" plywood must be applied to each shear panel. Since fiberboard sheathing is used it is assumed that the sheathing is not continuous past the second floor and therefore the wall cannot be considered as a single length for overturning. Because of the height of the wall above the sliding door, materials such as stucco would allow the wall to be considered as a single length, removing the requirements for hold-downs. A similar condition can be developed using fiberboard if the material is spliced at the top plates at the first-floor wall and blocking is installed at the second floor at the horizontal splice 8'-0" above this point. If this were done, no hold-downs would be required. As it is, the height is 8'-0", the total length of the two segments 13'-0" and the difference in load is 734 pounds. The uplift is therefore 451.7 pounds, which requires two BO framing anchors at each end of each wall segment.

First Floor Wall C

First floor interior walls in split-level construction nearly always receive a large amount of load and it can be anticipated that plywood will be needed for these walls in virtually every split-level house design. Several factors contribute to this requirement. First, this type of house generally has a long, narrow two-story section with a garage at either the front or the rear. Both the geometry and the size of the garage cause a large area to contribute load in proportion to wall length. In addition, the wall at the front of the garage is usually a relatively short wall, requiring that the tributary area multiplier be greater than 1.25. Finally, the general configuration normally results in only one or at most two interior cross walls at the first floor level of the two-story portion.

Despite the factors noted, wind load is still the controlling factor in this particular design. Sill bolting must be closely spaced and the wall itself must have 3/8" Structural II plywood applied. If it is assumed that the second floor joists span 21'-0" parallel to the wall relatively little vertical load contributes to overturning resistance. The wall without hold-down anchors does not come close to figuring. The vertical load on the wall is 95 pounds-per-foot and the 100-pound-per-foot hold-down graph for HD #1 has been used. The readings on this graph are therefore slightly higher than the actual conditions and if the allowable horizontal load were less than the actual load the actual allowable horizontal load should be interpolated. If, for instance, the 11'-4" wall were acting alone the allowable load of 4,200 pounds would be slightly less than the 4,247 actual load. Since this allowable load is for a 100-pound-per-foot vertical load it is not possible to determine whether the overage exceeds 5% and the next larger hold-down is required. In the actual case, the 11'-4" wall is assisted by the overturning capacity of the 7'-0" wall and the total horizontal load allowed is much greater than the actual load. The refinement of this interpolation is therefore unnecessary.

First Floor Wall E

There are advantages and disadvantages in providing a shear wall adjacent to a garage door opening, as opposed to using the Special Garage Front Wall Detail. The primary advantage is that a lower multiplier may be used for the interior shear walls when a shear wall is adjacent to the garage door. The major disadvantage is that a short shear wall, such as occurs in Model C, has a very heavy load applied to it and requires a large hold-down anchor and its attendant reinforced grade beam. Although the length of the two-story portion varies between models a comparison can be made between Model B and Model C to assist in determining which type of design appears to be more economically feasible in a given part of the country. In this particular instance the shear of 770 pounds-per-foot requires three 5/8" round sill bolts in the 4'-0"-long wall plus the installation of hold-down #3 at each end. This type of installation, while possible, is not very practical. To install both the hold-downs and the bolts, the three bolts must be placed at 12" o.c. in the center 2'-0" of the wall, leaving approximately 7" between the sill bolt and the hold-down bolt. Hold-down bolts should never be considered as resisting shear. When an installation of the type required for this wall is detailed it is suggested that the plans also require the contractor to provide a template (such as a precut sill plate) to assure proper placement of sill and hold-down bolts.

First Floor Walls G and H

Because of the fiberboard siding it is again necessary that the wall be considered as one story high when selecting overturning lengths for these walls. The stud portion of Wall G is only 4'-0" in height and is therefore able to provide a large amount of horizontal load capacity for overturning purposes. At wall H the high vertical load of 526 pounds-per-foot allows the 19'-4" wall length to be capable of resisting overturning. No further checking is therefore needed. Note that at Wall G the shear material varies and the total shear capacity of the various combinations of materials must be checked. In this instance the shear material on the exterior is fiberboard, the interior portion of the mid-level adjacent to the garage has gypsum board on one side, and the interior portion adjacent to the family room has gypsum board on two sides. The shear values for each of these materials times their respective wall lengths must be added to determine the total shear capacity of the wall.

CALC FORM 9

Job EXAMPLE HOMES Model 'C'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 175	Overturning
MID -	LEVEL									
B	65	13.0	15	1268	1080	12.0	105.7	1/2@6	OK	L=14' OK
D	65	13.0	"	1268	1680	13.0	129.2	1/2@6	OK	180 FRNG. ANCH.
	w=	237#/,		L=9' -	1020 + $\frac{31}{100}$(1500-1020) =				1198	
				4'					237	
							2x	100 =	200	
									1635	
			U =	$\frac{8}{13}$(1680-1635) = 277#						
E	8.67	15.75	15	2048	1727	21.0	97.5	1/2@6	OK	OK
				L=15.6	-1500+					
				5.6	- 190+					
				2x100	= 200					
					1890					

CALC FORM 9

Job EXAMPLE HOMES Model 'C'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 175	Overturning
HIGH ROOF										
A	7.83	24	15	2819	1380	21.0	134.2	—	OK	L=21' P_{ALL}=2740
E	"	"	"	2819	1380	12.5	225.5	—	3/8" ST II 8d@6	L=21' OK
									100 MIN	
G	6.08	10.5	"	958	1380	40.0	34.5	—	OK	L=48' OK
									175	
H	"	"	"	958	1380	27.25	50.6	—	OK	L=20' 4" MAX OK
1ST STORY WALLS										
A	17.0	12.08	"	3080	2134	13.0	236.9	1/2@39	3/8 ST II 8d@6	2 BOERING ANCHS
			L=9'-0"	-1000						
			4.0'	- 200		$U = \frac{8}{13}(2134-1400) = 451.7$#				
			2x100	= 200						
				1400						

CALC FORM 9

Job EXAMPLE HOMES Model 'C'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 200	Overturning
1ST STORY	WALLS									
C	170	2383	15	6077	4247	18.33	331.5	½@2.6	3/8" STR II 8d@4,12	HD#1
			L=11'-4"	- 800	TRY	HD#1 -	4200			
			7'-0"	- 310			2330			
				1110 -	NG		6530			
									175	
E	170	12.08	15	3080	2134	40	770	3 ⅝"φ	3/8" STR II 8d@4,12 2 SIDES	HD#3 . P_{ALL}=2520
G	15.25	10.5	"	2402	4697	42.0	111.8	½@6	OK	L=24'; H=4' OK
	SHEAR	MAT: -	28x17	5 + 4.5	x100	+ 9.5 x	200 =	4900 +	450 +	1900 = 7250 - OK
H	15.25	10.5	15	2402	3660	33.17	110.3	½@6	OK	L=19'-4" OK

MODEL D

As stated in Chapter II–3 the fact that most walls extend to the roof and are therefore of variable heights leads to considerable complexity in the design of Model D. Most of the complexities occur not at the lower level of the two-story portion but rather at the mid- and second-story levels.

The framing of the three previous example homes is fairly obvious and no framing plan has been provided. For the reader to understand how the vertical loads have been determined for the mid-level and high roof of Model D the exposed beam framing has been indicated on Figure 2.35A. Because of the many walls used as shear walls the vertical load calculations become lengthier. Due to the fact that the walls are of variable heights each calculation also takes slightly longer; the average height of each wall must be determined. In the example shown, these heights have been calculated but in actual practice the elevations and sections should indicate most of these wall heights.. Scaling each wall at its approximate horizontal center is acceptable for determination of wall heights.

Mid-Level Wall A

When variable height walls are utilized the deflection of walls of the same length would be different under the same amount of loading. This is the same as saying that the deflection of a cantilever beam with a concentrated load at its end would vary as the length of the cantilever varied. In addition, the presence of adjacent wall above and below openings can completely alter the deflection characteristics of a given wall. So many factors enter into the determination of the actual deflection conditions for a particular situation that it is not possible to present formulations for the determination of each possible combination. The method presented is reasonably accurate in many cases and has the effect of raising the maximum shear-per-foot considered in all cases.

For the mid-level Wall A the method is not correct in that the fixity offered to the piers on either side of the kitchen window by the high section of wall above and below the window would allow these two piers to deflect less than the shear wall at the corner of the house. The method shown for determining shears is therefore almost exactly the opposite of the probable actual conditions. The important point, however, is that the shear-per-foot for which the walls must be designed is raised from approximately 108 pounds per foot to 120 pounds per foot. Calc Form 9 (Model D, Mid-Level Wall A) indicates the method for making this determination. The width of each individual shear wall is first divided by its average height. The resultant ratios are added together, then each ratio is divided by the summation of the ratios. In this case the first wall is 4'-0" wide and 8.68' high for a ratio of 4.0/8.68 = 0.4608. The sum of the ratios for the three walls is 1.1455 and each ratio is then divided by this summation to obtain, in the case of the first wall, a second ratio of 0.4023. If done correctly, the second series of ratios should add up to 1.0. The total seismic or wind load (whichever governs) is then multiplied by each succeeding ratio to obtain the load to each successive shear wall. The load should then be divided by the length of the shear wall and the maximum shear obtained for any wall should be used in the design of all walls along the particular line of resistance being considered. In the case of Wall A all three walls are 4'-0" in width and so the maximum shear is 478/4.0 = 119.5 pounds-per-foot. This shear is well below the allowable for the shear material being used and therefore design is not affected. For overturning, the shear walls on either side of the kitchen window can be considered as one length since there is ample wall above and below the window. This length is

EXAMPLE HOMES - MODEL 'D' - VERTICAL LOADS/FOOT

MID-LEVEL

LINES A & E

W = ROOF - 5.25×12 = 63
WALL - 8×20 = 160
223#/1

LINE B

W = ROOF - 1.6×12 = 19
WALL - 11.37×20 = 227
246#/1

LINE H

W = ROOF - 8.17×12 = 98
WALL - 8×20 = 160
258#/1

LINE I

W = ROOF - 11.17×12 = 134
WALL - 10.4×20 = 208
342#/1

LINE J

W = ROOF - 9.33×12 = 112
WALL - 12.34×20 = 247
359#/1

HIGH ROOF

LINES A & G

W = ROOF - 4.75×12 = 57
WALL - 8×20 = 160
217#/1

EXAMPLE HOMES-MODEL 'D'-VERTICAL LOADS/FOOT

HIGH ROOF

LINE C

W = ROOF - 4.89 x 12 = 59
WALL - 9.5 x 20 = 190
249 #/'

LINE F

W = ROOF - 4.78 x 12 = 57
WALL - 9.1 x 20 = 182
239 #/'

LINES K & M

W = ROOF - 7.25 x 12 = 87
WALL - 8.0 x 20 = 160
247 #/'

LINE L

W = ROOF - 10.5 x 12 = 126
WALL - 10.5 x 20 = 210
336 #/'

1ST STORY WALLS

LINES A & G

W = ROOF - 4.75 x 12 = 57
2ND FLR - 1 x (17.5 + 20) = 38
WALL - 17 x 20 = 340
435 #/'

EXAMPLE HOMES - MODEL 'D' - VERTICAL LOADS/FOOT

1ST STORY WALLS

LINE D

w = 2ND FLR - 1.33×37.5 = 50
WALL - 8×20 = 160
210#/1

LINE K

w = ROOF - 7.25×12 = 87
4.33×12 = 52
2ND FLR - 5.25×37.5 = 197
WALL - 14.5×20 = 290
626#/1

LINE M

w = ROOF - = 87
2ND FLR - = 197
WALL - 17×20 = 340
624#/1

14'-0" and a check of the "No Hold-Downs Required" graph for a vertical load of 200 pounds-per-foot indicates that the wall is stable.

Mid-Level Wall B

This wall would normally have lath and plaster on each side and therefore have a shear capacity of 200 pounds per foot. The actual shear is 258.5 pounds per foot and plywood must be used. The wall has a vertical load of 246 pounds per foot and an interpolation is necessary in order to determine the maximum allowable horizontal load for overturning. This proves to be insufficient and the uplift formula must be applied in order to determine the uplift load of 1,560 pounds. This is within the capacity of the strap type hold-down as shown on Detail 25/4.

Mid-Level Wall E

In this case the adjustment for wall heights is quite accurate and probably reflects very nearly the actual shears developed by wind or seismic load. The short wall is 4'-0" in length and 8.68 feet average in height for a ratio of 4.0/8.68 = .4608. The longer wall is 9'-0" long and 12.05' average height for a ratio of 9.0/12.05 = 0.7469. The total of these two ratios is 1.2077. Each ratio is then divided by the summation of the ratios to obtain the secondary ratios of .3816 and 0.6184. Each is then multiplied by the total load of 1,695 pounds which results in the load to the 4'-0" wall of 647 pounds and the load to the 9'-0" wall of 1,048 pounds. It will be found that the lowest wall always develops the highest shear, in this case 161.8 pounds-per-foot in the 4'-0" wall. This shear should be used in the design of <u>both</u> walls. Horizontal load capacity proves insufficient without hold-downs and the uplift formula must be applied. Since the load at the 4'-0" wall indicates an uplift of 742 pounds, two BO framing anchors should be used at each end of each wall. The 9'-0" wall is checked for uplift only to determine which of the two walls governs. The wall with the highest shear does not always have the highest uplift load so each wall should be checked in all cases. The same hold-down anchors should always be applied to each wall section along any line of resistance.

Mid-Level Walls H, I and J

Although each of these walls is a different height they do not act together and so each can be designed separately. One of the disadvantages in designs of this type is exemplified by Wall I in that it is the shortest of the shear walls but receives the largest load. It will be recalled that the discussion on page II–51 indicated that walls I and J were not required to be considered as shear walls. This design indicates why interior shear walls, when they are used, should be made as long as other design considerations permit.

In the case of Wall I the situation described requires the use of a strap type hold-down. At Wall I the total height is 10.4' and the height of the doorway adjacent to the shear wall is 6'-8". The opening is therefore approximately 65% of the total height and the wall may not be extended. At Wall J the total height of the wall is 12.34' with the same height opening. The opening is therefore 6.67/12.34 = .54 or 54% of the total height and the wall may be considered 14'-4" long for purposes of overturning.

High Roof Walls

These walls need to be designed in the same manner as indicated for the mid-level. The highest walls are not nearly as high as those at the mid-level and most prove to be sufficient for both shear and overturning. The door in the wall at Line C does not allow the wall to be considered as a single unit for overturning. The 11'-0"-long segment does prove to be sufficient, however, and no hold-downs are required. At Line F no adjustment for wall height has been made as each wall is approximately the same height. At Line G the assumption has again been made that the openings at the wall are windows rather than sliding doors. If the openings were sliding doors hold-downs would obviously be required and either Detail 27/4, 36/4 or 38/4 would be required to be used for the attachment to the garage header.

First Floor Wall A

At this wall the length of sill is different from the length of shear wall available. In the sill bolt column the sill length has been entered and the shear-per-foot for this length calculated. The sill bolt spacing is then determined from this shear-per-foot rather than the shear-per-foot in the shear walls themselves.

First Floor Wall D

Because the Special Garage Front Wall Detail has been used at the front of this structure this wall is required to be designed to carry 100% of the seismic load for the two-story portion. The resultant shears and overturning require a very heavy design for this wall; 3/8" plywood must be utilized on both sides with a #3 hold-down required at each end of each section. Because of the high shear in the wall the actual number of sill bolts required has been noted rather than the sill bolt spacing. In checking for the hold-down required it is reiterated that the best method to be used is to consult the 200 pounds-per-foot vertical load graph for hold-downs #1, #2 and #3 in succession until the total horizontal resistance offered by the two walls is greater than the actual load. Consulting the graph for hold-down #3 indicates that a 12" x 18" grade beam with two #6s is required for any length of wall. In some instances, where two walls such as these occur, the size grade beam or amount of reinforcing required might vary from one length of wall to the other. When the two walls are as close as these the largest size grade beam or heaviest reinforcing should be used throughout, with the reinforcing extending beyond the farthest end of each wall as indicated by the "a" dimension shown for the reinforcing steel used.

First Floor Line G

Since the Special Garage Front Wall Detail is used along this line no calculation of shear or wall design is required.

First Floor Lines K and M

Because of the length of these walls no special considerations need be made. The shear-resisting material along Line K again varies. The minimum shear resistance offered has been noted under the column Shear Material. Since the shear-per-foot is less than the minimum shear allowable the total horizontal load resistance of the wall need not be calculated as was the case for Model C.

CALC FORM 9

Job EXAMPLE HOMES Model 'D'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
MID-	LEVEL									
A	11.0	6.0	15	990	1187	12.0	119.5 MAX.	½@6	OK	L=14' H=11.77'
	WALL ADJUST.		$\frac{4}{8.68}$ =	0.4608	÷1.1455=	0.4023	x1187=	478		P_{ALL}=1650+
			$\frac{4}{10.79}$ =	0.3707	" =	0.3236	" =	384		
			$\frac{4}{12.74}$ =	0.3140	" =	0.2741	" =	325		
				1.1455		1.0000		1187		
									200	
B	11.0	12.75	15	2104	2670	10.33	258.5	½@3'-4"	⅜" ST II 8d@6,12	H=11.37'-STRAP HD
		P_{ALL}=10	00+$\frac{46}{100}$	(1550-	1000)=	1253-	NG			
		U=$\frac{11.37}{10.33}$	(2670-	1253)=	1560#- STRAP HD					
									180	
E	11.0	7.25	15	1196	1695	13.0	161.8 MAX.	2-½"Φ 3-½"Φ	OK	L=4' H=8.68' 205
	WALL ADJUST.		$\frac{4}{8.68}$ =	0.4608	÷1.2077=	0.3816	x1695=	647	(161.8)	L=9' H=12.05' 790
			$\frac{9}{12.05}$ =	0.7469	=	0.6184	=	1048	(116.4)	2x100=200
				1.2077		1.0000		1695		1195
		U=	$\frac{8.68}{4}$	(647-305)=		742				2 BO FRMG. ANCHS.
			$\frac{12.05}{9}$	(1048-890)=		212				

CALC FORM 9

Job EXAMPLE HOMES Model 'D'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
MID-	LEVEL									
H	4.5	6.0	15	405	1703	21.0	81.1	½@6	OK	L=15'-6" P_{ALL}= 2850
									200	
I	5.33	11.12	"	889	1790	6.75	265.2	2-½"ϕ	3/8" ST II 8d@6,12	L=6.75' H=10.4'
	$P_{ALL}=650+\frac{42}{100}(875-650)=745$#					U=	$\frac{10.4}{6.75}(1790-745)=1610$			STRAP HD
J	8.15	9.75	15	1192	1421	9.0	157.9	2-½"ϕ EA PANEL	OK	L=14.33' H=12.34'
										P_{ALL}=2370#
HIGH	ROOF								180	
A	6.37	7.75	"	741	883	8.0	110.4	—	OK	L=21' OK
									200	
C	6.37	14.58	"	1393	1537	18.33	83.9	—	OK	OK
		L=11.0'	H=9.5'	$1300+\frac{49}{100}(1800-1300)=$				1545		

CALC FORM 9

Job EXAMPLE HOMES Model 'D'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 200	Overturning
HIGH	ROOF									
E	6.37	14.25	15	1362	1501	13.0	115.5	——	OK	180 FRMG ANCH.
	L=7.33	H=9.1	560+	$\frac{39}{100}$ (865-	560)	=	679			
	L=5.67	" "	350+	" (550-	350)	=	428			
					2×100	=	200			
							1317	$U=\frac{9.1}{13}$(1501-1317)=128.8		
									180	
G	.6.37	7.42	15	709	848	4.5	188.4	——	OK (4.7% over)	L=21' OK
									100	
K	4.5	5.25	"	354	1269	38.0	33.4	——	OK	L=44' OK
									200	
L	5.0	10.5	"	788	2230	24.33	91.7	——	OK	L=12.5 H=10.5 P_{all}=2350#
									180	
M	4.5	5.25	"	354	1269	38.0	33.4	——	OK	L=44' OK

CALC FORM 9

Job EXAMPLE HOMES Model 'D'

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
1ST STORY WALLS										
A	15.54	11.0	15	2564	2891	8.0	361.4	13'-222.4 ½@4-0	3/8" ST II 8d@4,12	OK
		L=10.0	P= 2500 +	$\frac{35}{100}$	(3125 −	2500) =	2719			
		L= 4.0	400 +	〃	(500 −	400) =	435			
							3154			
D	15.54	33.0	15	7692	11936	18.33	651.2	7-0 4-5/8 11-4 6-5/8	3/8" ST II 8d@4,12 2 SIDES	HD #3
		HD #3	11-4	8875						
			7-0	5300						
				14175						
									180 MIN	
K	14.50	14.63	15	3182	6609	37.0	178.6	½@5.0	OK	L=44' OK
									180	
M	14.50	10.50	15	2284	5968	37.33	159.9	½@6	OK	L=44' OK

ZONE 2 CALCULATIONS – WOOD FRAME SHEAR WALL DESIGN

Although Calc Form 8 has been used for the Zone 2 calculations, the form is applicable for any area not requiring wind load designs.

Calc Form 9 should be used in Zones 2 and 3 when wind load design is required.

As shown by the following calculations very few conditions lead to loads that require additional shear-resisting material, sill bolt spacing other than minimum, or hold-down anchor installation. In the few instances where that statement is not correct, the conditions are somewhat unique and the same condition would prove to be insufficient for the vast majority of houses with similar configurations.

CALC FORM 8

Job EXAMPLE HOMES   Model "A"-ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 160	Overturning
A	267	27.5	9.7	½@6	OK	L=27.5' OK
B	1210	24.83	48.7	½@6	OK	L=17.33' P_{ALL}=1860
C	195	4.0	48.8	½@6	OK	180+100=280 OK
D	1047	30.5	34.3	½@6	OK	L=45.5' OK
E	944	13.5	69.9	½@6	OK	L=15' P_{ALL}=2750+
F	1410	25.17	58.0	½@6	OK	L=12' P_{ALL}=1638
G	207	9.0	23.0	½@6	OK	L=15' P_{ALL}=1400

CALC FORM 8

Job EXAMPLE HOMES　　　　　　Model 'B' - ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 330	Overturning
HIGH	ROOF					
A	986	24.0	41.1	——	OK	L=28' OK
B	686	24.0	28.6	——	OK	L=28' OK
E	686	18.5	37.1	——	OK	L=36' OK
I	986	20.0	49.3	——	OK	L=36' OK
LOW	ROOF				230	
C	262	18.0	14.6	½@6	OK	L=18' OK
D	308	4.0	77.0	"	OK	1 BO FRMG ANCH.
	P_{ALL}=178	+100=	278	U=$\frac{8}{4}$(	308-2	78)=60#
H	241	20.67	11.7	½@6	OK	L=20.67' OK
1ST S	TORY	WALLS			330	
A	2556	24.0	106.5	"	OK	L=28' OK
B	2229	18.5	120.5	"	OK	L=18.5' P_{ALL}=4250
					250	
E	1047	18.0	58.2	"	OK	L=18' OK
					200	
F&G	1849	19.0	97.3	"	OK	L=15' P_{ALL}=7000

CALC FORM 8

Job EXAMPLE HOMES Model 'B' - ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 330	Overturning
1ST STORY WALLS						
I	1562	9.0	173.6	2-½"Φ EA. PANEL	OK	2@4.6-2×500=1000
						2×100= 200
	U= $\frac{8}{9}$(1562-1200)=328.9					1200
						1 BO FRMG. ANCH.

CALC FORM 8

Job EXAMPLE HOMES Model 'C' - ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 175	Overturning
MID-	LEVEL					
B	540	12.0	45.0	½@6	OK	L=14' OK
D	840	13.0	64.6	"	OK	OK
	L=9.0' -	P_{ALL} =	1020 +	$\frac{37}{100}$(1500	-1020	)=1198
F	864	21.0	41.1	½@6	OK	L=15.5' P_{ALL}=1500+
HIGH	ROOF					
A	690	21.0	32.9	——	OK	L=21' OK
E	690	12.5	55.2	——	OK	L=21' OK
G	690	400	17.3	——	OK	L=48' OK
H	690	27.25	25.3	——	OK	L=20.33' MAX. OK
1ST STORY	WALLS					
A	1067	13.0	82.1	½@6	OK	L=9.0 - 1000 4.0 - 200 OK
					200	
C	2124	18.33	115.9	"	OK	180 FRMG. ANCH.
	L=11.33	- 800				
	7.0	- 310	U =	$\frac{8}{18.33}$(2	124 - 13	10) = 3566
	2×100	= 200				
		1310				
E	1067	4.0	266.8	2-½"Φ	3/8" ST II 8d @ 6, 12	HD #1 P_{ALL}=1300#

CALC FORM 8

Job EXAMPLE HOMES Model 'C' - ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 125 min	Overturning
1ST STORY WALLS						
G	2349	42.0	55.9	1/2@6	OK	L=24' H=4' OK
H	1830	33.17	55.2	1/2@6	OK	L=19.33' OK

CALC FORM 8

Job EXAMPLE HOMES Model 'D' - ZONE 2

SHEAR WALL DESIGN

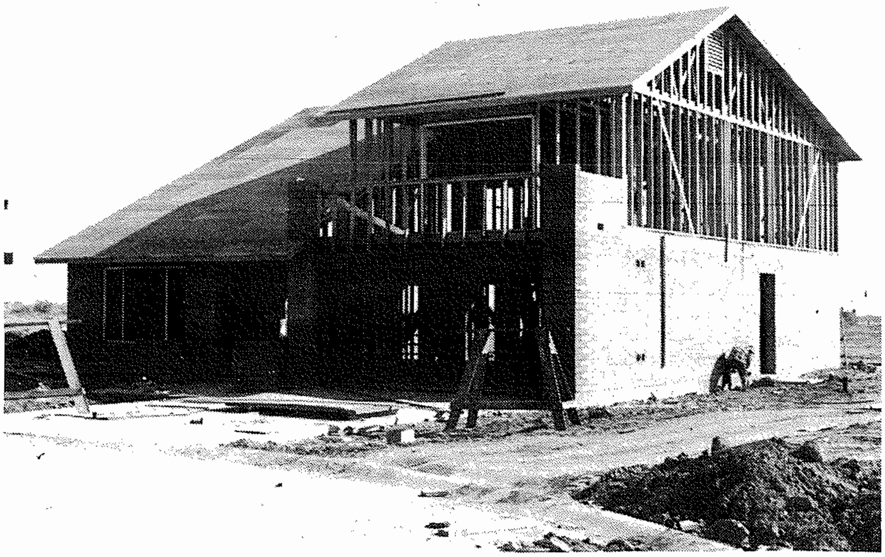
LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
MID -	LEVEL					
A	394	12.0	59.7 max	1/2@6	OK	L=14', H=11.77'; P_{ALL}=1650
	0.4023	x394	=239			
					200	
B	1333	10.38	128.6	1/2@6	OK	OK
	H=11.37'	P_{ALL}=	$1000+\frac{46}{100}(1550-1000)=1253+100=1353$			
					180	
E	848	13.0	80.9 max	1/2@6	OK	L=4', H=8.68' 205
	0.3816	x848=	323.6	(÷4 =80.9)		L=9', H=12.05' 790
						995
H	852	21.0	40.6	1/2@6	OK	L=15.6' OK
					200	
I	895	6.75	132.6	2-1/2"φ	OK	1 BO FRMG ANCH.
	H=10.4	$P_{ALL}=650+\frac{42}{100}(875-650)=745+100=845$				
		$U=\frac{10.4}{6.75}(895-845)=77$#				
J	711	9.0	79.0	2-1/2"φ EA PANEL	OK	H=12.34', L=14.33' OK.
HIGH	ROOF				180	
A	442	8.0	55.3	——	OK	L=21' OK
					200	
C	769	18.33	42.0	——	OK	H=9.5, L=11.0 P=1300+

CALC FORM 8

Job EXAMPLE HOMES Model 'D' - ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material	Overturning
					200	
HIGH	ROOF					
F	751	13.0	57.8	——	OK	OK
	H=9'	L=7.33	-560+			
		5.67	-350+			
			910+			
					180	
G	424	4.5	94.2	——	OK	L=21' OK
					100	
K	635	38.0	16.7	——	OK	L=44' OK
					200	
L	1115	24.33	45.8	——	OK	H=10.5'; L=12.5' OK
					180	
M	635	38.0	16.7	——	OK	L=44' OK
1st STORY WALLS						
A	1446	8.0	180.8	13'- 111.2 1/2@6	OK	L=10' P_{ALL}=2500+
					200	
D	5968	18.33	325.6	7'-2 5/8@4 11'4" 3 "	3/8" ST.II 8d@4,12	HD #1
	HD#1	11-4	4950+			
		7.0	2650+			
			7600+			
					180 MIN	
K	3305	37.0	89.3	1/2@6	OK	L=44' OK
					180	
M	2984	37.33	79.9	1/2@6	OK	L=44' OK



CHAPTER II–5 MASONRY EXTERIOR WALLS – SHEAR WALL DESIGN

The design of houses with exterior masonry walls differs little fröm the design of houses with wood stud walls insofar as use of the Calc Forms is concerned. The Methodology restricts the use of masonry exterior walls to one-story construction or to the first floor of two-story construction. Designs with higher masonry walls must be engineered. Example house Model E is shown on the following pages and is very similar to Model A in floor and roof plans.

In the determination of shear walls the major difference between masonry and wood framing is that a masonry wall need be only 2'-0" in length to qualify as a shear-resisting element. The same rules apply in determining the length of wall for overturning purposes as apply for wood stud walls. In some parts of the country steel lintels are used over window and door openings and reinforcing is omitted from these lintels. When this is done the unreinforced lintel is prone to crack away from the remainder of the masonry wall and hence supply neither fixity nor partial fixity at the top of the wall. Lintels supported in this manner should therefore be considered a part of the opening when determining whether the wall qualifies for increased length for overturning.

FIGURE 2.38. **Steel lintel beams are used in lieu of reinforcing steel in some locales. These beams are acceptable for vertical load but are more prone to earthquake damage.**

Tributary areas are determined in the

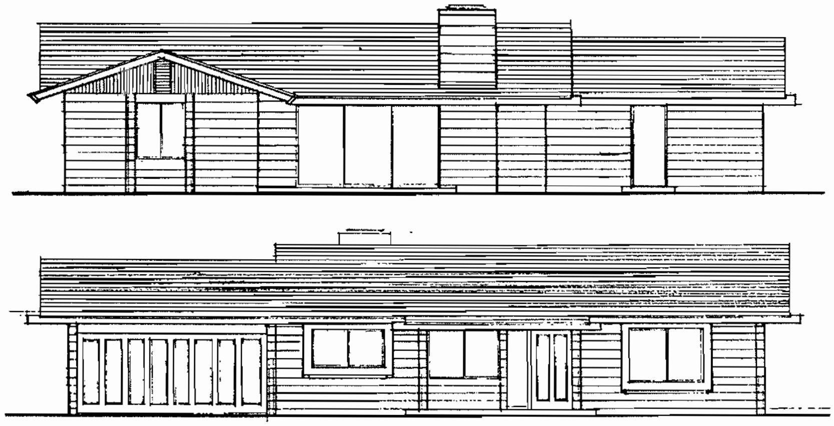


FIGURE 2.39. Front and Rear Elevations – Model E.

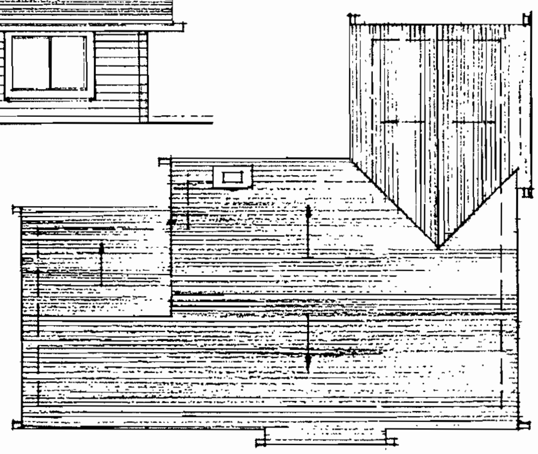


FIGURE 2.40. Roof Plan – Model E.

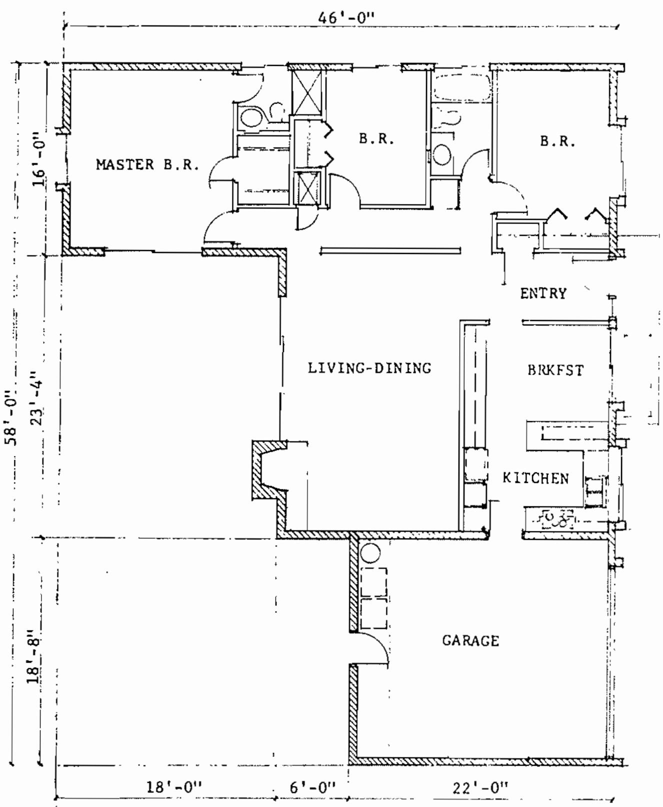


FIGURE 2.41. Floor Plan – Model E.

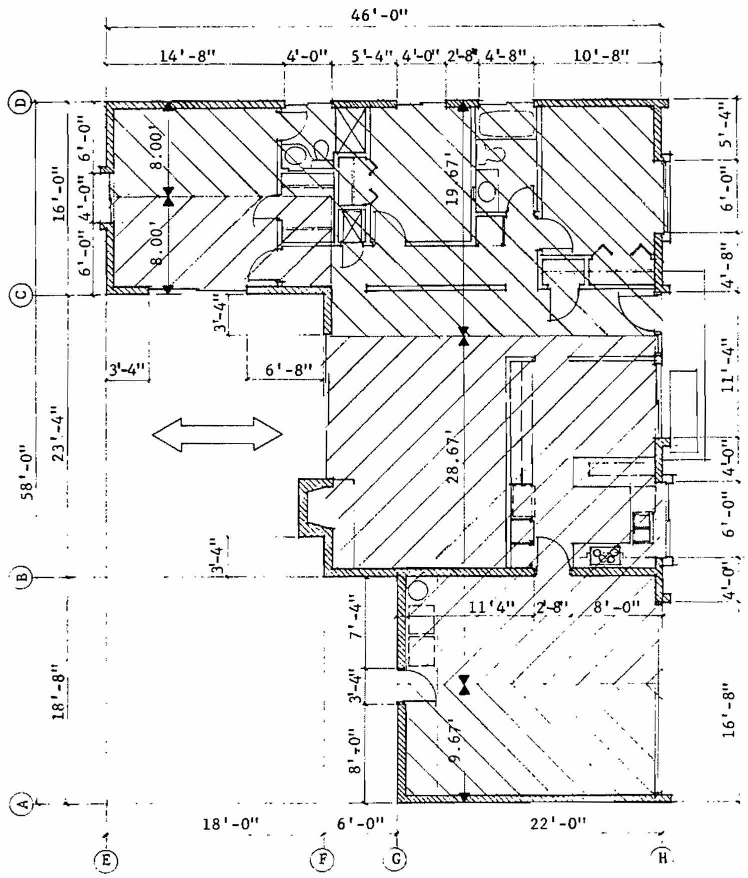


FIGURE 2.42. Tributary Areas, Transverse Direction – Model E.

same manner as they are for the models already discussed. Since masonry walls are quite rigid it is not necessary to use a multiplying factor when interior masonry walls are used at the first floor of two-story construction. The masonry walls should be designed to take the entire load, however, and all interior wood frame walls should be ignored. All masonry walls should be designed. There is no Special Garage Front Wall Detail presented for masonry walls.

Should a partially grouted masonry wall prove to be unstable in overturning it may be more economical to solid-grout the wall rather than provide a grade beam. If this is done the wall must, of course, be checked to determine that the solid grouted wall is stable.

The elevations, roof plan, floor plan and tributary area plans for Model E – as stated, very similar to Model A – are presented in Figures 2.39, 2.40, 2.41, 2.42 and 2.43. Differences in design from the design for Model A are discussed in relation to the applicable Calc Form.

Calc Form 1

The top of Calc Form 1 is filled out in exactly the same manner as it would be for a wood-frame house. The only real difference, in fact, is the determination of load for the exterior

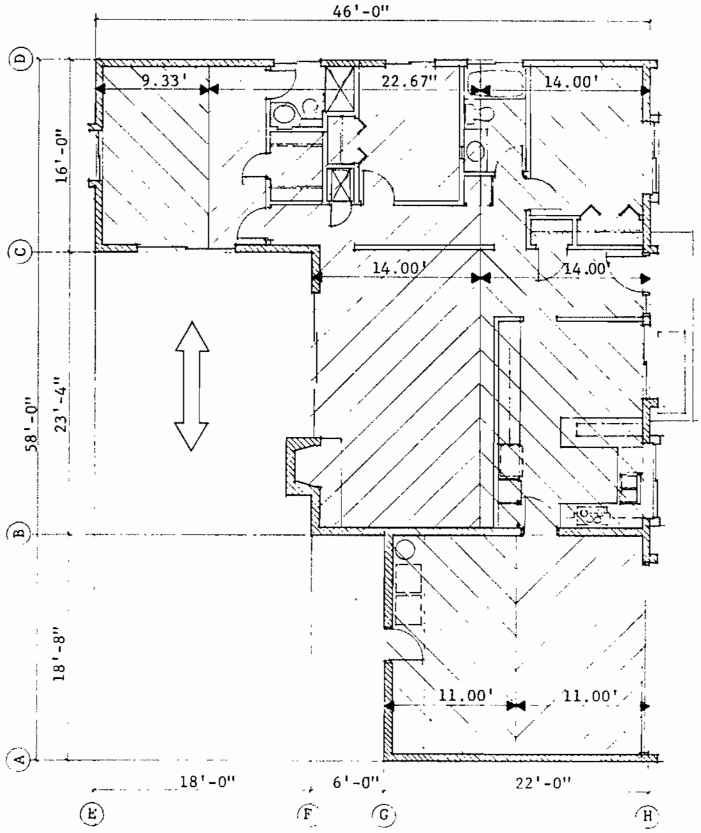


FIGURE 2.43. Tributary Areas, Longitudinal Direction — Model E.

wall. Table 3.2 indicates the weight of concrete block of various thicknesses, depending on the spacing of the grouting of the wall. Brick masonry weight should be calculated as 10 psf per inch of thickness. In this instance the material is concrete block and the reinforcing designed for Zone 3 is placed at 48" o.c. vertically. Eight-inch block therefore weighs 46 psf. It is assumed the wall is furred and dry wall installed on the interior of the block. The weight of the furring and the dry wall is noted to obtain the total load of 49 psf for the exterior wall. To obtain the equivalent seismic unit load from Table 3.6 wall weight of 49 psf may be divided into segments of 30 and 19 psf. This yields equivalent seismic unit loads of 2.250 + 1.425 = 3.675 psf. This is entered under the "ceiling" load determination at the bottom of Calc Form 1. Interior wall loads using 10 psf for wall weight are again obtained from Table 3.4 and a total "ceiling" load of 4.562 psf is obtained.

Calc Forms 6 and 7

The determination of tributary areas and seismic loads is the same for masonry houses as it is for wood frame houses. Fireplace load is added, even though masonry exterior walls have been assumed.

Calc Form 8

To use Calc Form 8 properly it is necessary to become familiar with Figures 3.17 through

CALC FORM 1

Job EXAMPLE HOMES　　　　Model 'E'

ONE-STORY RESIDENCE
ROOF, CEILING AND WALL WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	CEILING	Material	Actual Weight	Seismic Weight
Roofing	DECORATIVE ROCK	6.0	0.800	Framing		2.0	0.267
Sheathing	1/2" PLYWD	2.0	0.267	Finish	GYP. BD./ ACSTC FIN.	3.0	0.400
Framing	TRUSSES	2.0	0.267				
TOTAL		10.0	1.334	TOTAL		5.0	0.667
			Unit Load				Unit Load

EXTERIOR WALLS	Material	Actual Weight	Material	Actual Weight	INTERIOR WALLS	Material	Actual Weight
Framing	FURRING	1.0			Framing		4.0
Finish	GYP. BD	2.0			Finish	GYP BD	2.0
Finish					Finish	" "	2.0
	CONC. BLK	46.0					
TOTAL		49.0			TOTAL		8.0

	"CEILING" UNIT LOAD
Ceiling	0.667
Exterior Walls	3.675
Interior Walls	0.250
TOTAL	4.592 psf

ROOF UNIT LOAD: 1.334 psf

VENEER LOAD: ________ lb/ft

CALC FORM 6

Job EXAMPLE HOMES Model 'E'

TRIBUTARY AREAS

	ROOF AREA		CEILING AREA	
LINE	Length x Width	Area	Length x Width	Area
A	26.0x11.67	303	22.0x9.67	213
B	26.0x7.0+		22.0x9.0+	
	32.0x21.67	875	28.0x19.67	749
C	20.0x10.0	200	18.0x8.0	144
D	30.0x21.67+		28.0x19.67+	
	20.0x10.0	850	18.0x8.0	695
		2228		1801
E	20.0x11.33	227	16.0x9.33	149
F	20.0x6.67+		16.0x8.67+	
	43.33x16.0	827	39.33x14.0	689
G	18.67x13.0	243	18.67x11.0	205
H	43.33x16.0+		39.33x14.0+	
	18.67x13.0	936	18.67x11.0	756
		2233		1799

CALC FORM 7

Job EXAMPLE HOMES Model 'E'

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
A	303	1.334			213	3.675	404 783 1187
B F.P.	875	"		1167 600 1767	198 551	3.675 4.592	1767 728 2530 5025
C	200	"			144	4.592	267 661 928
D	850	"			695	"	1134 3191 4325
E	227	"			149	"	303 684 987
F F.P.	827	"		1103 600 1703	689	"	1703 3164 4867
G	243	"			205	3.675	324 753 1077
H	936	"			551 205	4.592 3.675	1249 2530 753 4532

CALC FORM 8

Job EXAMPLE HOMES Model 'E'

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material	Overturning
A	1187	22.67	52.4			L=22.67' OK
B	5025	25.33	198.4			L=17.33' P_{ALL}=5130
C	928	10.67	87.0			L=7.33 - 900
						3.33 - 180
						1080 OK
D	4325	34.0	127.2			L=46.0' OK
E	987	12.0	82.3			L=16.0' P_{ALL}=4380
F	4867	8.0	608.4			L=4.0'
				UNGROUTED -		2x270=540 - NG
				GROUTED -		2x590=1180 - NG
						2x2600=5200
						12x18 GRADE BM.
						2-#4 T&B - 'a'=11'-0"
G	1077	16.0	67.3			L=8.0'
						2x1080=2160 OK
H	4532	18.0	251.8			L=16.0 - 4380
						14.0 - 3320
						7700 OK

3.19. The first two graphs indicate the allowable horizontal load for solid-grouted masonry and concrete block masonry with cells containing reinforcing grouted, respectively. Model E is of concrete block construction with only the cells containing reinforcing grouted. Figure 3.18 is therefore used. This single graph presents all the information the designer requires to determine the adequacy of short walls for shear, bending, grade beam requirements, or the determination that no grade beam is required. The curve near the bottom part of the graph indicates the allowable horizontal loads for which no grade beam is required. Generally speaking, the loads in exterior masonry walls are so low that this curve will prove to be the only portion of the graph to which the designer need refer. The curve is extended for walls up to 22'-0" in length on Figure 3.19. The heavy dashed line above the other curves on Figure 3.18 indicates the maximum horizontal load each wall is capable of carrying in shear. For wall heights of 6'-0" or less this line should be used. When the wall height is 8'-0" or more, bending in the shear wall governs and the curves labelled for various wall heights must be used. Each curve has segments designated as requiring grade beams with varying amounts of reinforcing. Detail 29/4 refers to masonry as well as wood stud shear walls and the extent of the grade beam can be determined from that detail and Table 4.6.

WALL DESIGNS

Wall A

The allowable shear for walls of this type is 725 pounds-per-foot. This figure is actually based on partially reinforced walls requiring cells grouted at 8'-0" o.c. and is conservative for closer grout spacings. For Wall A it is apparent that less than 2'-0" of wall would be sufficient to resist the load of 1,187 pounds. Figure 3.19 indicates that a load in excess of 8,000 pounds can be taken by a 22'-8" long wall, so it is apparent that this wall is sufficient and requires no special design. The small wall elevation indicated in the upper left hand corner of Figure 3.18 specifies that one #4 minimum must be used at each end of each wall. This, again, is in accordance with the requirement for minimum reinforcing in partially reinforced walls. The wall elevation for fully reinforced walls, Detail 51/4, indicates that this type of wall must have a #5 at each end of each pier. For the load shown, then, this wall would be adequate in either Zone 2 or Zone 3.

Wall B

Inspection of the 8'-0" high curve on the graph indicates that a wall 6'-0" long would be sufficient for shear and bending. The No Grade Beam Required graph, Figure 3.19, indicates that the allowable horizontal load for a 17'-4" long wall is 5,130 pounds which is in itself sufficient.

Wall C

With a sliding glass door between the two sections of wall it is neccessary that each wall be checked individually for overturning. With no grade beam required the 7'-4" long wall is capable of supporting a horizontal load of 900 pounds, which is slightly less than the total load of 928 pounds. The 3'-4" long wall can support a load of 180 pounds, bringing the total allowable horizontal load to 1,080 pounds, so no grade beam need be used.

Wall D

The length of this wall plus the assumption that all windows are of a height that allows the wall to be considered continuous indicate that no further checking is necessary.

Wall E

This wall can be considered 16'-0" long for overturning purposes and is more than adequate. It should be noted that the shear-per-foot has been tabulated for each of the walls. This is not actually necessary since the use of the tables automatically determines if the wall is sufficient. In determining the sufficiency of the wall the curves indicating the use of grade beams should be used. As discussed under overturning previously the wall may be extended for purposes of overturning and the overturning length is not actually the length of shear wall available to resist the load. Even in Wall F, where the wall is quite short, the shear is below the allowable and so the only real checking usually necessary for masonry wall is that required for overturning.

Wall F

Since the wall adjacent to the fireplace will be built with the fireplace it can be considered 4'-0" in length. Checking the No Grade Beam Required curve, the two 4'-0"-long walls on this line are determined to be insufficient, however. Referring to the curve for 8'-0"-high walls, the allowable load is in excess of the actual load but a 12" x 18" grade beam must be used with two #4s top and bottom. In this instance Table 4.2 indicates that this reinforcing steel should extend 11'-0" beyond the end of each wall. The grade beam should be turned at Lines B and C and extended 11'-0". The grade beam should also be continuous between Lines B and C. When two walls are closer together than the required "a" distance as shown by the aforementioned figure it is not necessary to overlap the reinforcing or increase its size. In other words, if the reinforcing and grade beam extend continuously between two shear walls, the "a" distance need be applied only at the opposite ends of each shear wall. This is true for wood frame construction as well. Note that solid-grouted walls would also require a grade beam at this line of resistance.

Wall G

The two 8'-0"-wide sections are sufficient to resist overturning with no grade beam and, as discussed above, are automatically sufficient for both shear and bending.

Wall H

These relatively short shear walls are required to take a substantial load with the result that even though the wall on the right hand side of the plan is 16'-0" in length for overturning purposes, the allowable horizontal load for this section is slightly less than the actual load. Again, the two walls in combination are sufficient and no grade beam need be used.

† † † † †

CHAPTER II–6 MISCELLANEOUS STRUCTURAL CONSIDERATIONS

Although the design of shear walls is the most difficult, many other considerations are necessary to provide proper construction for seismic resistance. As stated in Chapter II–1 a continuous shear path must be supplied for loads generated by earthquake to be transmitted back to the ground. This chapter covers a variety of structural items, most of which have to do with that principle.

Basic Layout Principles

A house need not be large to be complicated. This is most aptly demonstrated by Figure 2.20. Now that the reader is more familiar with seismic and shear wall design, it should be apparent that such a house cannot be designed by the use of the recommended provisions in this report since no shear wall occurs at the front of the clerestory, and if the alternative rear wall is utilized, no shear wall occurs at the rear of the living room also. It is this type of basic error that this publication attempts to eliminate. This is not to say that the house as designed could not be constructed safely, but structural engineering would be required to provide a lateral load resistance system at the locations mentioned. The primary considerations for seismic resistance in basic house layout are shear walls, roof diaphragms and floor diaphragms.

The first step in the seismic resistive design of a house is to review the roof and floor plans to determine the boundaries of roof and second floor diaphragms. It is obvious from the previous chapters that it is necessary to provide shear walls at or near the edges of all portions of a house, including interior lines where broken diaphragms occur. It is essential that the designer recognize individual diaphragms and provide shear wall at their edges, whether they be roof or floors. It is also necessary that shear wall be provided near the edges of second floors that do not extend to the exterior walls. If loads are resisted in each of two perpendicular directions in a single plane, loads acting in all other directions in that plane will be resisted. If a diaphragm configuration exists such that shear wall cannot be provided at the exterior boundary in each of two perpendicular directions, engineering should therefore be provided.

Having concluded his inspection for required shear walls, the designer should also determine whether other walls in the structure should also be employed for shear resistance. Walls continuous to the roof framing will attempt to resist lateral loads and should be designed. These walls are therefore included in the Methodology. In addition, long interior walls on the first floor of two-story residences are also normally designed. Since shorter shear walls will automatically deflect more under less total load, the walls not designed will attempt to take some load but will be limited in their deflection by the deflection of the diaphragm plus that of the designed shear walls and will therefore not deflect a sufficient amount to incur damage. Judgmental decisions regarding the length of such walls have been largely removed from this report, and rules have been devised, instead, to allow the designer to determine when interior walls must be designed as shear walls.

Diaphragms

Roof diaphragms

It is not intended that this publication require all engineering procedures required for the seismic design of commercial buildings, but rather that it correct those deficiencies that have caused distress in houses subjected to earthquake. Very little damage has been noted to residential roof construction other than that caused by the failure of masonry fireplaces. For this reason, nonstructural diaphragms such as spaced sheathing have been allowed. This is not to imply that the roof construction need not be properly fastened, nor that the horizontal loads generated by the roof should not be given a shear path at the lines of resistance.

As stated previously, the gypsum board or plaster ceilings in most homes tend to act as diaphragms. Interior walls parallel with the motion receive the load from the ceiling and hence support some of the load of interior walls perpendicular to them. The roof in this instance is asked to carry only its own weight plus a portion of the weight of the exterior walls. For standard construction, any roof sheathing described in Table 3.2 of Part III is acceptable.

When all or most interior walls extend to the roof because of a sloping ceiling or exposed framing, the construction of a roof diaphragm can become more critical. The architectural requirements of that type of construction usually preclude such an event, but in no instance should spaced sheathing be used with that type of design. Straight sheathing is also not recommended, but may be acceptable for designs of that sort. It is much better if the sheathing can be installed diagonally or if plywood sheathing can be used with that type of construction.

Details 7/4 through 13/4 are intended primarily for use with split-level construction. They should be used, however, at all breaks in roof diaphragms. Detail 7 deals with framing perpendicular to the break in levels while the remainder of the details are for framing at the lower roof parallel to the stud wall. Detail 7/4 ties the two sections together for a pull of 100 lb/ft. The remaining details are each capable of resisting 800 lbs with the load per foot dependent on the spacing. Although the latter details are to normally be spaced at 8'-0" o.c. in Zone 3, secondary structures such as garages need only be tied at the exterior walls and ridge. When no ridge occurs a tie should be placed at approximately the mid-point between the exterior walls. In all cases, the roof should be tied to the abutting wall positively rather than with a ledger attached to the face of the stud and the rafters then attached to the ledger.

When sloping roofs with a ridge are encountered, the diaphragm is "bent." This would be similar to having a V-shaped web in the steel beam, which would be unacceptable since it would buckle. In roof systems, however, any tendency for buckling is resisted by the vertical load-carrying capacity of the roof and roof diaphragms can tehrefore be considered continuous across ridges and valleys as long as the various planes of the roof intersect. This reasoning cannot be extended to roofs (or floors) that have a vertical offset between levels or planes. In this case, the buckling would have to be resisted by bending of the vertical members between the diaphragms and is beyond the scope of this report. Such offsets can be treated, however, if each diaphragm is considered separately and provided with all the necessary lateral support.

Floor diaphragms

All floor diaphragms have been assumed to be 1/2" plywood or better, nailed with 10d nails at 6" maximum o.c. at all edges. Interior sheet nailing (field nailing) may be 10d at 10" o.c. Plywood to be used in a floor diaphragm should be a minimum of 2'-0" by 4'-0" in its minimum dimensions. The joints in one direction should be staggered the distance of one-half the plywood sheet. For a normal 4 x 8 sheet of plywood, this would imply that the joints were staggered 4'-0". No detail has been provided for this requirement, but the third and fourth diagrams under Table 3.10 indicate acceptable layouts of floor sheathing.

Where glued floor construction is utilized, nail spacing at the edges of sheets may be reduced to 10d at 12" o.c. Glued plywood is not acceptable for exterior surfaces, and may be used only for interior floors.

At stairway openings cut into second-floor diaphragms, the opening is usually the width of a single run of stairs. For this reason, a shear wall is recommended on one side of the opening, with the presumption that the shears developed on the other side are capable of transfer through the remaining portion of the diaphragm at either end of the stairwell opening. At split-entry houses, at stairways that require openings greater than approximately 4'-0" in width, and at any stairway or other opening whose length is greater than 50% of the parallel diaphragm dimension, it is required that such a shear wall be provided. At split-entry homes and other wide openings it is recommended that a wall be placed on each side of the "long" dimension of the opening whenever possible.

The diaphragm ratios required herein will normally automatically restrict shears in the diaphragm to values somewhat below those allowed for the nailing specified and no actual design is therefore required. Horizontal diaphragms may have whatever joist spacing is allowed for vertical load by the identification index of the particular plywood used. If the plywood is thick enough to span the required distance between floor joists and is nailed as mentioned above, it will normally be capable of resisting all shears transmitted through it.

Shear Resisting Materials

As stated in Part I, the large variety of materials and systems available make it impossible to consider all systems or even a variety of fastenings for a given shear material. The fastenings required by Table 3.9 and 3.10 are those most commonly used for shear resistance and their values have been determined by tests. Tests have been run on some materials with stud spacings of 24" and with studs smaller than 2 x 4. These have not been included, since the tests are selective and do not consider the broad spectrum of materials allowed. All shear-resisting materials in Tables 3.9 and 3.10 are assumed to be applied to 2 x 4 or larger studs at 16" maximum o.c., with the exception that 3/8" and thicker plywood may be placed on studs at 24" o.c. when the conditions of the footnotes on Table 3.10 are met.

Attachment

All finish materials should be structurally attached to the framing, whether the walls are shear walls designed in accordance with the previous chapters, or whether they are interior walls not so designed. It is not intended that the attachments indicated in the tables be used for the shear wall portions only. It will be recalled that the shear-resisting material is relied upon to carry the load to the sill bolts below windows, for instance. In addition it has been assumed by the loading established herein that interior walls will support their own weight plus the load developed by a portion of the cross walls framing into them. Non-designed interior walls need not have stud spacings or sizes as stipulated. The finish materials should nevertheless be attached as indicated.

The installation of plywood with 2" or 2-1/2" nail spacing is not recommended for residential construction, but does become necessary in a few instances. When such nailing is utilized, the width of the studs behind the plywood joints should be 3 x minimum. The use of double studs does not meet this requirement, since the member should be of one piece to provide shear transfer between the plywood sheets and of a width that will preclude splitting of the stud while nailing the plywood.

All shear-resisting materials that come in sheets should be of 2'-0" x 4'-0" minimum dimensions. Where odd dimensions (such as 21'-0") would normally allow the use of 4'-0" sheets, plus a single sheet narrower than 2'-0" in width, one of the 4'-0" sheets should be cut back in order to achieve a greater width on the last sheet. The one exception to this rule is that the width between openings is sometimes less than 2'-0". The shear material in this instance should be in one piece spread out to a minimum dimension of 2'-0" above and/or below the opening.

MPS attachment requirements

In Zones 2 and 3, the attachment of shear-resisting materials shall be in conformance with this report and other indications of positive methods of attachment as presented in the MPS shall not be used without the submission and acceptance of tests as stated under "Capacity of Shear Resisting Materials" in Chapter II—4.

Shear Transfer Details

The details presented in Part IV together with the nailing schedules are intended to provide a shear path for all loads from the point they are developed to the ground. In developing any special details the designer may wish to present to clarify a condition not shown in the details in Part IV, it will be helpful to think in terms of this shear path. This is especially true where shear-resisting materials are interrupted, such as at the second floor line, or near the foundation.

Sole plate connections at exterior and other designed shear walls

When exterior walls are connected to first-floor wood framing the connection of the sole plate shall be in accordance with the Nailing Schedule, Table 4.5. When connected through a slab on grade to an exterior foundation wall, 1/2" round anchor bolts shall be provided at a maximum spacing of 6'-0" o.c., except when the Calc Forms indicate a closer spacing is required. The maximum sill bolt spacing shall apply to the connection of either sole plates or mudsills to foundation walls, direct to foundations, to basement walls, retaining walls, knee walls, etc. Interior shear walls connected to a wood floor framing system having no continuous footing directly beneath shall be connected to the floor in accordance with details 21A/4 or 21B/4.

When an interior shear wall is non-bearing and does not otherwise require a footing, Detail 19/4 may be utilized for the installation of the required anchor bolts. This detail may not be used when strap or angle hold-down anchors are required.

Sole plate connections for non-designed interior walls

Interior walls not designed as shear walls shall have their sole plates connected to wood floor framing with a minimum of two 16d nails to each perpendicular floor joist or, as specified under Wall Framing, Table 4.5, to parallel joists. When the sole plate rests on a concrete slab, 1/2" round anchor bolts at 6'-0" o.c. may be installed as shown on Detail 19/4 or gunbolts may be used at 2'-0" o.c. as shown on Detail 20/4.

Sole plates to mudsill

In some areas the sole plate and mudsill are not the same member. Detail 18A/4 indicates this condition as an alternative detail. Although this is the only place at which this detail is shown in Part IV, it is intended that any time a sole plate is rested on a mudsill the nailing of the sole plate should be in accordance with Table 4.5. It is desirable, and is the usual practice, to extend the finish material to at least the bottom of the mudsill. In this circumstance the nailing of the shear-resisting material should be directly to the mudsill whenever possible. When this is done the condition noted in the footnote at the bottom of Table 4.5 may be assumed to apply; i.e., all nailing indicated may be half of that shown in the table except that sole plate nailing may not be less than 16d at 16" o.c.

Studs to sole plate

It is required that Detail 26/4 be used at all exterior corners of all wood frame houses in seismic Zones 2 and 3. That is the same detail used as the framing anchor hold-down detail. When the sole plate stops at the edge of the double stud (as it must in one direction) the 2-3/4" maximum dimension should also be 2" minimum. This bolt should not be considered part of the normal sill bolting required. In addition, in Zone 3 it is required that framing anchors be placed on the first two studs from the corner and, in two-story construction, at 4'-0" o.c. (on alternate sides of the wall) for the entire length of all exterior stud walls. These anchors serve a number of purposes. As stated in the chapter on inspection, if the nailing of shear-resisting material is deficient anywhere in the structure, that location can usually be discovered at the sole plate. The anchors tend to contribute some additional horizontal shear resistance at that location. In addition, they provide a better reaction for loads perpendicular to the wall. Most importantly, however, the connection of studs to sill plates proved to be one of the major weaknesses observed in the San Fernando earthquake and was virtually the only location at which the vertical component of earthquake motion played a major damaging role. The framing anchors provide a positive tie between the studs and the sill plates for upward loads. It should be noted that Detail 24/4 indicates that framing clips may be used in lieu of the framing anchors at the individual studs, but that the framing anchors must be used at the exterior corners.

Hold-Down Anchors

Framing anchor type

This type of hold-down is not used in commercial construction and has been devised for the report to provide for the minimal uplift loads often developed in residential construction. The connection depends not only upon the framing anchors installed at the double stud, but also on the nailing of the shear-resistant material to the anchored stud and to the sill plate. A notation is provided requiring that the sill plate used with this detail have an allowable fiber stress in bending of 1,250psi or better. Some of the materials that will provide the required strength are Select Structural Ingleman Spruce, Lodgepole Pine, Western White Pine, Ponderosa Pine, Red Pine, Spruce-Pine-Fir; #1 or Select Structural Balsam Fir, Eastern Spruce, Hem-Fir, Idaho White Pine, Mountain Hemlock, Sitka Spruce, or Western Cedars; #2 or better Redwood, Douglas Fir, Eastern Hemlock, Tamarack, Northern Pine or Southern Pine. Northern White Cedar and Eastern White Pine do not qualify for this requirement. The above classification is on the basis of stress only and should not be interpreted as implying that a given species is otherwise acceptable as sill plate material, particularly with regard to termite or decay prevention. It is recommended that strap hold-downs be used instead of angle hold-downs for walls less than 8'-0" in length wherever possible.

Strap type hold-downs

In addition to the hold-down shown on Detail 25/4 many commercial strap hold-downs are available. Oftentimes the value for these hold-downs is listed with a 1/4 increase in nail or bolt values allowed for metal straps, and with a notation indicating that a further increase of 1/3 may be taken in using the strap in conjunction with lateral loads. This publication does not consider these increases as cumulative. Since the values listed are normally listed with the 25% increase, the 1/3 increase allowable for horizontal load has not been taken. It is suggested that the designer use the values shown for commercial hold-downs, whether the allowed increase is 25% or 33%, but that the cumulative increase of 1.25 x 1.33 = 1.67 not be taken. Since uplift is calculated for the installation of strap type hold-downs, other commercial hold-downs with different vertical capacities may be substituted when justified by the use of the uplift formula.

Angle hold-downs

The angle hold-downs as shown on Detail 34/4 provide a maximum in hold-down capacity. Similar pre-fabricated hold-down anchors are commercially available and may be substituted.

Numerous graphs are provided to make the designer's work easier in determining the size hold-down required. The uplift formula is not applicable for this type of hold-down since the bolt is located approximately 5" inside the end of the wall.

Special conditions

Occasionally hold-downs are required at second floor walls in two-story residences. Where a wall occurs directly below, the double stud required above should be repeated at the first floor, and the strap or angle type hold-down should be connected from the base of the second floor wall to the top of the first floor wall. The typical hold-down detail can then be provided at the bottom of the double studs in the first floor wall framing. When this double stud occurs at the end of the wall on the first floor also, the hold-down load at that location will be considerably greater and may require a larger hold-down than was used above.

Overturning at two-story high shear walls

All the graphs and formulas presented assume a one-story-high shear wall. When the wall is two stories in height (of the type shown in Figure 1.1) the formula shown in Section 6.5 of Part III will convert the horizontal load to the equivalent horizontal load applied to a ten-foot-high equivalent wall.

Using that formula, seismic and wind loads should not be intermixed. To avoid possible misinterpretation of the conditions for which the formula is applicable (see Section 8.5), a shear wall is considered herein to be two stories high only if it receives little or no support from adjacent shear-resisting material at the second floor line. None of the walls in Models B, C, or D are considered to be two-stories high for purposes of overturning but all the walls shown in the above referenced figure would be so considered.

Struts

Very little mention has been made of the use of struts other than that the continuous top plates of walls collect and carry load along the line of resistance to the shear wall. In addition, struts should be used whenever the major portion of the diaphragm that contributes load to a wall is not in direct contact with the top plates of the wall or a continuous extension of those plates. An example of a condition such as this would be Figure 2.17. It will be recalled that the exterior wall at the wing on Line C was not used to resist lateral loads from the main portion of the house. Even though the interior wall is in line with the exterior wall and the top plates are probably continuous, the interior portion of the wall was not considered in the design as shear wall since this portion does not extend to the roof line. Had the exterior wall been longer or if it had been decided to design the interior wall as shear wall, tributary areas for this line of resistance would then be as shown on the figure. The main portion of the roof (and therefore the main portion of the load) would then not be in contact with the top plates of the wall. In such an instance it would be desirable to connect the roof truss to the top plate of the wall in the manner shown on Detail 39/4. This is not to imply that this detail represents the load that would be developed in this particular strut; it is only used as an example.

Occasionally, when partial second floors are used, an interior wall not entirely under the floor is included as resistance for the floor. It is usually best to avoid this situation if the overlap of wall and floor is short but, to maintain the diaphragm ratio requirements, it cannot always be avoided. Examples for figuring strut requirements in this instance are contained in the Design Methodology, Section 8.1E.

Struts are frequently used in commercial buildings for conditions other than those described above — frequently in connection with short shear walls capable of resisting high loads. The only situation comparable in residential construction is the interior wall at the first floor of two-story construction. A diaphragm passing over an interior wall may be thought of in the same manner as two abutting beams resting on a single post. Thus, although allowable shear in a floor diaphragm (nailed as specified in this report) is slightly less than 300 pounds-per-foot, this pertains to load coming from one side of the shear wall only. The shear in the wall could therefore be as high as 600 lb/ft without necessarily overstressing the diaphragm — provided one half the load came from each side of the shear wall. The nailing of the diaphragm to the blocking or joist over the shear wall would be overstressed, however, as would the toe-nailing or framing anchors to the top plate. To avoid the use of struts when shear in an interior wall exceeds 300lb/ft, the following recommendations are made:

Detail 5A/4 — Nail sheathing to blocking with 10d nails @ 3" o.c. and connect blocking to top plate with framing anchors at 2'-0" o.c. in seismic Zone 2 and 12" o.c. in seismic Zone 3.

Detail 5B/4 and 5D/4 — The joist will act as a strut. Connect to top plates as above.

Detail 5C/4 — Nail sheathing to top plates with 10d @ 3" o.c. When the top plates of the wall are longer than the shear panel because of an adjacent door or other opening, these plates will act as a strut. The shear-per-foot to the plates, considering their total length, should therefore be calculated in order to determine whether the above recommendations need be followed.

Chords

Chords, like roof diaphragms, were rarely observed to suffer any damage in the San Fernando quake. Because of the large number of interior walls in a house, the walls in any given direction cut the span of the chords down to the point where two 2 x 4 top plates, properly spliced and nailed, are adequate for virtually any condition. For this reason, Detail 22/4 is shown with a minimum length between splices of 8'-0". This dimension is not nearly as critical as the nine 16d nails between splices. It should be remembered that the chords also act as struts and for this reason the minimum nailing should be adhered to from one end of the chords to the other. For bending alone, the chord would need only its maximum nailing near the center of the distance between perpendicular walls.

Wood Framing

Cutting and notching

Because each individual stud is required to act as a beam during seismic disturbances, the indiscriminate cutting and notching of such members is highly inadvisable. Restrictions on these procedures are spelled out in detail in the Design Methodology.

Cripple walls

In some instances short stud walls are provided below the first floor wood framing of residences. This condition is indicated in the top elevation of Figure 2.31. This type of construction could also be achieved by extending the foundation wall up to the bottom of the wood floor framing. If wood stud cripple walls are used below the floor framing instead, it is imperative that shear-resisting material extending down to the mudsill be attached to these walls. Because walls of this sort are usually solid in houses with no basements, it is not necessary that the wall be "designed" so long as the shear-resisting material is continuous. In some instances masonry veneer is used, particularly at the front of houses such as the one referred to above. It is in such instances that there is a tendency to ignore the requirement for shear-resisting materials. This proved to be a fatal error in a number of homes that suffered severe damage in the San Fernando earthquake (see Chapter I—2). When cripple walls occur as the upper portion of knee wall construction (see Knee Walls), the wood frame portion of the walls frequently contains window openings. In this case the basement is either developed, or designed to be developed, for future living space, and the walls should be designed as if they were the first floor walls of two-story construction.

Let-in braces

Diagonal 1 x 4 or 1 x 6 let-in braces are frequently used to brace construction before finish materials are applied. While they serve that purpose well, they are of little value in resisting lateral load and have hence been given no credit for seismic or wind resistance in this publication. Due to knots in the lumber, buckling of the brace, and/or failure of the nails at either the base or the top, these braces tend to give more of a false sense of security than they do actual resistance.

Knee wall construction

This type of construction, known by different names in various parts of the country, is shown on Detail 18/4. Frequently the knee wall does not extend around all four sides of the residence due to sloping grades. As long as the shear-resisting material is extended down to the sill plate at the top of the masonry or concrete wall, or to the top of the foundation on the sides where such wall does not exist, proper lateral resistance will be provided. In most cases any wall which will function properly as a retaining wall will also function in a stable manner during an earthquake. No special requirements are therefore set forth for the retaining wall itself since the construction requirements for such walls vary considerably from place to place. Although the wall is shown with no reinforcing and with an ordinary footing rather than a retaining wall footing, this and other similar details should not be interpreted as an endorsement of the kind of construction shown. Requirements of local building codes should, of course, be met in the construction of the masonry or concrete portion of the knee wall.

Basements

The same comments made with regard to the retaining wall portion of knee wall construction apply to basement walls as well. Basements usually extend, however, up to the first floor framing. Detail 16/4 indicates various methods of construction for these walls. The connection to the floor framing system is adequate to provide horizontal support at the top of the wall for minimal retaining wall action (30 pound equivalent fluid pressure) as well as for lateral load provided the vertical dimension between the basement floor and the top of exterior earth does not exceed 7'-6".

Foundations

For the purposes of this discussion two types of foundations are described. Stem-wall footings are shown on Detail 14/4 and consist of the footing, which should always be of concrete, and the footing wall which may be of concrete or masonry. Simple or trench type continuous footings are as shown on Detail 15/4 and are normally used under slabs on grade.

No requirements vary more from one location to another than those for foundations. The footings shown herein are fairly typical in many locations but are not intended to represent any of the variable conditions that apply to local areas, such as depth to frost line, water table, softness of soil which requires piles for support, etc. The footings as shown are to be considered minimal to meet seismic requirements only. For stem wall footings it is assumed that the stem wall is a minimum of 6" thick for one-story construction and 8" thick for two-story construction. The footings are assumed to be a minimum of 6" deep and 12" wide for one-story and 8" deep and 16" wide for two-story construction, and to be 12" minimum below undisturbed natural ground for one story, and 18" for two stories. All footings supporting wood framing are assumed to extend 6" above finished grade. Allowable soil bearing is asumed to be 1,000 psf. As indicated above, no footings are required under interior non-bearing shear walls, but either Detail 19/4 or 20/4 must be used for these walls at slab on grade floors.

Grade beams

The grade beams required to be used in conjunction with hold-down anchors are assumed to be poured monolithically. No construction joints should occur in their length.

It is acceptable, although not desirable, that a construction joint be placed at the end of the grade beam. It is much better practice to continue the footing for some distance beyond the grade beam in order to provide additional safety.

If the stem wall footing as shown on Detail 32B/4 or 33B/4 is used it is necessary that the same cross-sectional area be developed as is provided in the more normal grade beam cross-sections. This is necessary to provide the weight required to resist the hold-down. In achieving this cross-sectional area it may be desirable to deepen the footing. The previous chapter shows how to reduce the amount of reinforcing in the grade beam when this is done.

Slabs on grade

Floor slabs on grade provide frictional resistance during earthquake movements, and are utilized along with the footings to transfer the load developed in the structure back to the earth. For this reason it is best from a seismic viewpoint if slabs on grade are poured monolithically. Control joints are necessary for other reasons, however, and should be placed in the slabs as described by the MPS or local standards.

One of the principal problems created by earth motion, insofar as slabs on grade are concerned, is the movement of the slab away from the footing. This is particularly true when a plastic waterproof membrane is run continuously between the footing and the slab on grade. It is necessary to obtain a positive tie between the footing and the slab in order to prevent this type of damage. Any of the three details shown as Detail 15/4 will assist in rectifying this situation and one of the three must be used in Zone 3.

Prevention of footing cracks

It is fairly normal practice in house construction to extend pipes through the footings in a somewhat indiscriminate manner. Cracking due to either settlement or earth motion can be minimized by separating such pipes so that there is a minimum of 6" of concrete between them, by placing them at or near the center of the height of the footing, and by placing them through sleeves. Better still, of course, is to locate these pipes crossing below the bottom of the footing – through a sleeve and encased in concrete. Because of the critical nature of grade beams, it is essential that care be taken in placing pipes through such beams. Since normal footings are frequently unreinforced, they have a greater tendency to crack and the requirements for care in this condition are less critical seismically but more apt to occur. Therefore, both conditions described should be given careful thought by the designer, builder and inspector.

Garages

When the wall containing the garage door is not in line with, or near a line of resistance of the house, the wall on either side of the garage door must be counted upon to resist its share of the load. Since the walls are frequently very narrow, special garage front wall details are provided for both one- and two-story construction. The one-story garage detail is not only required for attached garages, but must be used for detached garages as well. When 4'-0" or more of wall is present on one side of the garage door or the other, the wall should be designed in lieu of the special detail. Where the special detail is used, it should be provided on both sides of the garage door. The one-story garage detail was designed using a garage 20'-0" in width and 25'-0" in depth with a 6 in 12 roof slope. Zone 3 seismic load and 15psf wind load were assumed. Roofing was assumed to be Spanish tile and all walls were assumed to be veneered with 4" masonry. Despite these very conservative assumptions in determining seismic load, wind load was found to govern because of the steepness of the roof. A 4 in 12 pitch ro f would allow the detail to sustain a wind load of 18.4 pounds-per-foot and a 3 in 12 pitch, a load of 20.8 pounds-per-square-foot. Since the garage is assumed larger than normal, the detail is assumed to be sufficient for virtually any garage in Zone 3 or wind zones requiring loads up to 20 psf. The Detail is restricted to the support of garage load alone and because it is conservative, may be considered as acting in lieu of a shear wall; i.e., capable of supporting the garage load tributary to it.

The two-story garage detail is somewhat less conservative. Model C is assumed as a typical condition, and in this case seismic load was found to govern for Zone 3. In Zone 2 a 15psf wind load was used. The above information will allow the designer to determine if the special garage front wall detail is applicable to his particular area. Unlike the one-story condition, this detail is primarily intended to restrict rotation of the second floor diaphragm. Although shear resistance is provided, the amount of load the construction will actually resist will be dependent on the length of the garage wall opposite the doors and the rigidity of the second floor diaphragm. For these reasons, the area normally tributable to this detail must be assigned to the rear wall of the garage in order to assure that all loads are adequately resisted.

These designs are not intended to imply resistance to wind uplift or other such forces.

Masonry

The masonry wall elevations shown on Details 51/4 and 52/4 indicate the required reinforcing for 8" nominal thickness, fully reinforced masonry walls and partially reinforced masonry walls respectively. Their attendant sections indicate the placement of this reinforcing. The partial reinforcing is deemed adequate to resist seismic load in Zone 2, but should be used only where local offices find it acceptable for the prevailing conditions in that area. While some areas of the country allow 6" concrete block construction, no charts or reinforcing steel details are presented for this type of construction since it is relatively uncommon. Similar details can be provided for this condition by local engineers. Details 53/4 through 57/4 indicate the ties required at the top of masonry walls to foor or floor framing. These ties are adequate for loads parallel to the wall in virtually all conditions and for loads perpendicular to the wall for walls up to 8'-8" in height (when measured above the first floor line). When walls are higher than this or when the distance from the floor line to the footing exceeds 4'-0", an engineer should be consulted.

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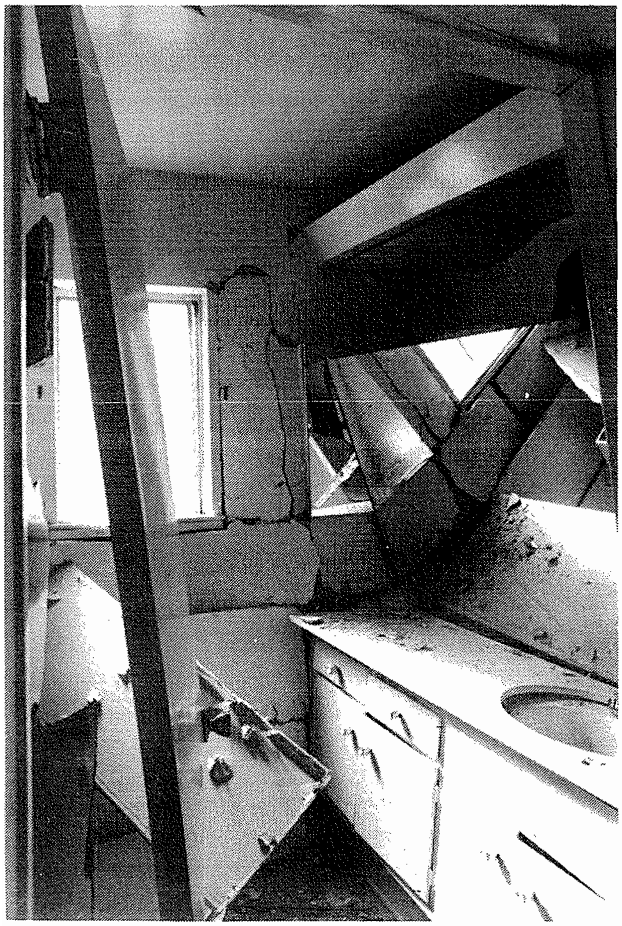

CHAPTER II–7 NON-STRUCTURAL ITEMS

If adequate shear walls are provided at all locations in a given residence, the types of motion causing significant damage to most non-structural items should be considerably reduced. In providing proper shear walls, the designer has therefore automatically reduced damage to many other materials as well. Nevertheless, some non-structural items can and do cause problems in earthquakes irrespective of the performance of shear walls. This chapter discusses those items most frequently damaged together with the corrective action to be taken.

Glazing

Reduction in extent of motion should normally prevent serious damage to window and door frames. Glass breakage cannot be entirely prevented but several things can be done to understand and attempt to prevent this type of damage. In houses utilizing shear-resisting materials other than plywood, much damage can be avoided by allowing at least 1/8" clearance between the jamb and the edge of the glass on each side of each pane. This type of installation is presently not uncommon and therefore costs nothing other than the designer's time to specify this clearance. When hold-down anchors or plywood sheathing are used, however, the deformation in shear walls is apt to exceed 1/8". The angle type hold-downs are subject to slippage of the bolts, with the result that as much as 1/8" deformation in a 4'-0" wide panel, 1/16" in 8'-0" wide, etc. can be caused by this alone. This would be in addition to the normal deformation of the shear material itself. When 3/8" or thicker plywood is used, deformations of more than 1/8" are to be expected if the shear resisting material is subjected to its full load capacity. If a 4'-0" wide plywood shear panel were used, glass placed in the same line of resistance could theoretically require 1/4" clearance for the plywood deformation plus 1/8" for the hold-down anchor for a total of 3/8". It is not always practical to achieve this much clearance at each window jamb and each individual case must be considered separately to determine its practicality. Items to be reviewed in making this determination would include the hazards involved, frequency of earthquakes (relatively high in Zone 3, and lower in Zone 2), expected maximum intensity of earthquakes in a given locale, any additional cost of the type of glass installation proposed, and the predicted glass replacement cost should the detail not be utilized. For these reasons the 1/8" clearance at jambs is required but larger clearances are left to the discretion of the designer. For other reasons, requirements for glass clearances are set forth in the MPS (Section 608-5) and recommended clearances are tabulated in the MAP (same section).

Damage to Cabinets and Bookshelves

Damage to kitchen cabinets is of two types – damage to the cabinets themselves and damage to the contents caused by either the cabinet attachment failing or the doors opening. While cabinets torn loose by earthquake forces can usually be re-hung at a cost which is not prohibitive, the additional cost to install the cabinets with screws instead of nails is also quite slight. Some installers, in fact, already use a method similar to that shown on Detail 50/4. For the cabinet itself, damage to contents can only be prevented by requiring positive locking devices on the doors. These devices are recommended; particularly for cabinets above the work counter level. While the cost of this type of device is somewhat greater than the systems presently used, the protection offered the homeowner is not only protection of the contents but also reduction of the danger of injury when these cabinets open and the contents fly through the room. These considerations make the cost of these devices seem minimal. Designers and contractors are encouraged to consider positive locking devices for all cabinet doors. Bookshelves and other full-height cabinets should be fastened in the same manner as kitchen cabinets detailed.

Fireplaces

Details 47/4 and 48/4 and the Design Requirements thoroughly spell out the methods for reinforcement and the requirements for tying masonry and other fireplaces to the structure. Heavy masonry fireplaces are basically incompatible with wood frame structures and it is therefore imperative that the strap ties be installed correctly and be attached to the framing. It is also essential that the designer be aware of the heavy load thrown into the structure by masonry fireplaces and that this load properly be accounted for in the design of shear walls.

Stone or other veneer attached to fireplaces creates additional problems, not only by increasing substantially the weight of the fireplace, but also by its propensity to separate from the firebox and/or chimney and fall during heavy seismic disturbances.

Because of the heavy loads involved, the strap ties are in all but one case bolted to tie members which extend a considerable distance into the structure in order to spread the load through the ceiling or floor. In the case of floor ties where the floor joists are perpendicular to the strap, the strap is shown as nailed to the blocking between the floor joists. Two 16d nails per block should be considered the minimum nailing required, as shown on Detail 48/4. Straps of this nature, when attached to the bottom of the joists, frequently cause the blocking to rotate out from between the joists when the strap is subjected to heavy tensile loads. For this reason, framing anchors are required to be applied at the end of each block if the strap is attached beneath the blocking. If the strap were not installed at the center height the holes for the strap would need to be drilled in a portion of the floor joists such that they would be weakened. For this reason, the only choice other than installing the strap as shown is to install it on the bottom of the blocking. When floor joists are parallel with the strap, the strap should be connected to a floor joist with two half-inch bolts per strap.

Veneer

Masonry or stone veneer creates a considerable problem in earthquake areas. The ties required by the Design Methodology are somewhat heavier than the requirements were in major earthquake areas during the time of the San Fernando earthquake. When masonry veneer separates from its supporting members, the resulting pieces can become dangerous missiles to anyone within a short distance of the veneered walls. While masonry or stone veneer is beneficial both for appearance and ease of maintenance, previous chapters indicate that it does add substantially to earthquake load. In some cases additional shear material or hold-down anchors will be required because of these additional loads.

The two principal differences between previous practice and the requirements as set forth herein are that ties are required at somewhat closer spacing than many areas of the country presently require, and that an air space between the backing and tied veneer is not allowed. This space must be poured solid with grout.

Mechanical Equipment

Water heaters

Gas-operated water heaters and furnaces can prove to be extremely dangerous during earthquakes since broken couplings create an immediate fire hazard. Many major earthquakes are accompanied by fires caused by failures of this sort. Figure 49/4 indicates a suggested method of attachment of most styles of water heaters to the wall structure. Although no such prefabricated angle tie presently exists, it is to be hoped that the commercial market will quickly fulfill this need. Similar ties can be fabricated to order for a given installation, although contractors might find it cheaper to have these ties fabricated from 2-1/2 x 2-1/2 x 3/16 angles. These ties are required whether flexible or rigid couplings are used at the top of the heater. It would seem that rigid connections would serve to supplant the angles shown. Experience has indicated this not to be the case since the water pipes are rather loosely attached in the wall and do not offer the water heater the type of support needed.

Heater legs also frequently crumple and therefore it is necessary to provide some anchoring at the base to prevent the heater from starting to move. While the particular detail shown need not be used, an equivalent method of tying must be provided for all water heater installations when the heater is higher than the least dimension (or diameter) of the base.

Floor furnaces

Furnaces usually have a low center of gravity, since the blower and burner units are normally mounted near the base of the unit. For this reason their propensity to overturn is less than water heaters. A positive tie should be affected between the furnace and its base, however, in order to keep the furnace from sliding. For furnaces with higher centers of gravity, ties should also be installed near the top. These ties can be created by fastening plumber's tape to the walls and wrapping it around the furnace or in a manner similar to that shown for water heaters.

Roof coolers

Evaporative coolers used in dry, arid regions present a greater problem than furnaces since the center of gravity of the equipment is only slightly below the center of its height. Clips or other mechanical fastenings must be used to connect the cooler to its base. The base itself should be anchored mechanically to the roof.

Gas shut-off valves

These valves must be provided for each home and should be labeled and located such that they are easily accessible to the homeowner.

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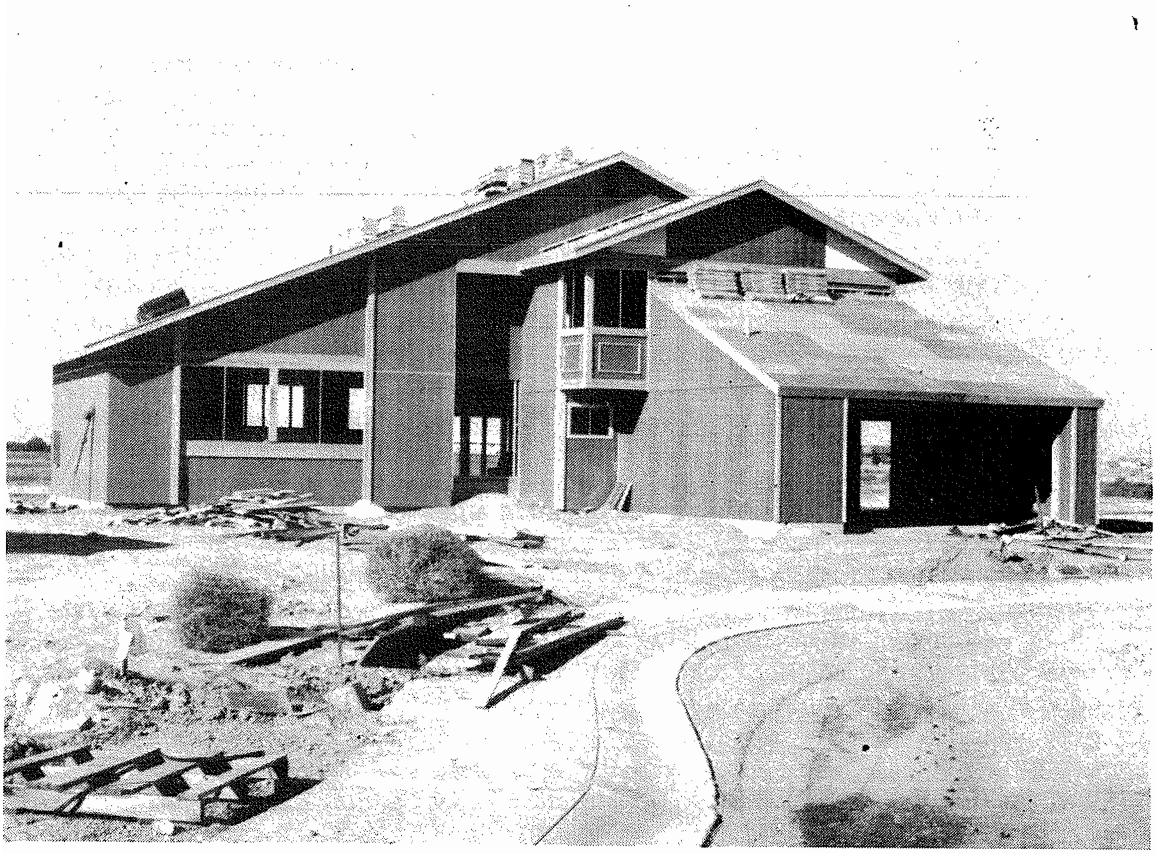


PART III SEISMIC DESIGN METHODOLOGY

SECTION 1: METHODOLOGY AND DEFINITIONS

1.1 Methodology

A. The methodology reflected by the criteria presented in Part III is intended to provide seismic designs for one- and two-family, one- and two-story dwellings similar to designs that would be developed using a rational engineering analysis. Nothing in these criteria shall be interpreted as prohibiting the use of an engineered design.

B. All allowable shears and other allowable loads specified in this report are for seismic or wind loads only and should not be interpreted as being applicable to either vertical or other horizontal forces acting alone.

C. Wind and earthquake loads need not be assumed to act simultaneously.

1.2 Definitions

Definitions presented here describe and discuss terms associated with seismic design which are not necessarily familiar to readers otherwise conversant with house construction terminology. The definitions are presented as they apply to subsequent sections of this report and do not necessarily incorporate the technical differentiations required for application to larger or more complicated structures. Some of the terms have been developed for descriptive use in this report only.

Anchor Bolt — Plain bar threaded at one end with 90° bend at the other. Length measured from end of threads to bend. Used to connect wood or steel to masonry or concrete.

Back-Span — As used in this report, back-span is associated exclusively with cantilevered diaphragms and is the distance from the line of resistance directly adjacent to the cantilever to the parallel line farthest away. To determine the line farthest away, only lines of resistance occurring in the diaphragm immediately behind the cantilever are considered. Back-span distance is normally the distance between the exterior walls in that portion of the diaphragm directly adjacent to the cantilever.

Chord — "Flanges" of a diaphragm when it functions as a beam supporting horizontal load. Normally the staggered top plates of the exterior walls in wood frame construction perform this function, as does the uppermost horizontal reinforcing bar in masonry walls.

Combination One- and Two-Story Construction — See Section 2.3D.

Corners (See Figure 3.1)

Exterior Corner: Intersection of two exterior walls which, when viewed in plan, has the smaller angle (usually 90°) inside the house.

Interior Corner: Intersection of two exterior walls which, when viewed in plan, has the smaller angle (usually 90°) outside the house.

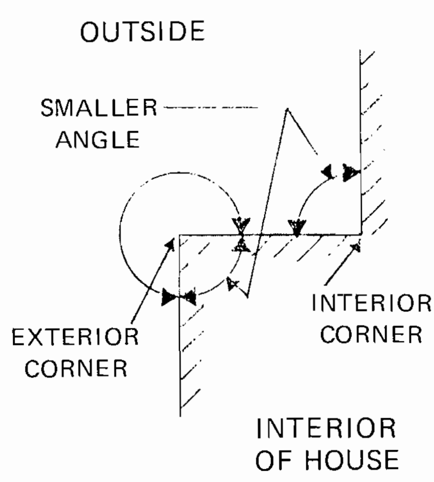


FIGURE 3.1. Exterior and interior corners.

Cripple Stud Wall — Wood stud wall less than one full story in height.

Diaphragms — A wood diaphragm is a composite unit consisting of a shear-resisting material applied to members of sufficient size to prevent buckling, plus structural members at its sides capable of resisting the tension and compression developed when it acts as a beam to resist lateral loads. All floors and roofs must qualify as diaphragms. (A wood stud wall is a vertical diaphragm but is called a shear wall.)

1.2 Diaphragm (Cantilevered) — Second-floor or roof diaphragm overhanging the line of resistance (normally the exterior wall) below it.

Diaphragm Depth — Occasionally used as a synonym for diaphragm length.

Diaphragm (Floor) — All floor diaphragms are assumed to use 1/2″-thick plywood (minimum) nailed per Table 4.5 as the shear-resisting element. For plywood installation, or use and installation of other materials, see Section 8.1B. For plywood grades or other methods of fastening, see Section 6.3C3.

Diaphragm Length — For the direction of loading being considered, the dimension between the chords of the diaphragm. Normally the dimension between the exterior walls (as viewed in plan) perpendicular to the width. Diaphragm Length should not be confused with shear wall length or the length used for determining tributary area.

Diaphragm Ratio — Ratio of the distance between two adjacent lines of shear resistance to the least depth of the diaphragm between them (width:depth).

Diaphragm (Roof) — Because of the lack of damage to these elements in earthquakes coverings such as straight and spaced sheathing are allowed for roof diaphragms despite the little shear resistance they provide. For restrictions on such usage see Section 8.1A. Roof diaphragms installed in accordance with Table 4.5 and other Sections as are applicable to the material used need not be further investigated or designed.

Diaphragm Width — For the direction of loading being considered, the dimension between lines of resistance parallel to the load. In cases of more than two such lines, each section of diaphragm (between lines of resistance) is considered as an independent unit. Thus, one diaphragm continuous past several lines of resistance will have differing widths for each segment.

Discontinuous Roof — See Section 2.3B.

End Truss — Truss located directly over the end wall of a gable-roofed house. The truss is considered a part of the shear wall. Shear-resisting materials must be applied to the truss.

Equivalent Seismic Weight — Horizontal design force generated by one square foot of a single material as it is installed in a structural system. Equivalent Seismic Weights of all materials in a horizontal structural system are added together to obtain the Unit Load for that system. Table 3.2 gives Equivalent Seismic Weights for various actual weights.

Force (Seismic Force) — Used synonymously with Seismic Load.

Grade Beam — Continuous footing adequately reinforced to allow sufficient and proper resistance to the overturning generated in a shear wall located along its length. As used in this connotation, the soil beneath the grade beam may be fully capable of providing vertical load support to the building.

Hold-Down Anchor — Mechanical fastening device used to transmit the tension developed by overturning in a shear wall from the wall to the structure below.

Length — See Diaphragm Length, Shear Wall Length, "Solid" Wall Length, and Tributary Area Length.

1.2 Line of Resistance — Line along which one or more shear walls occur. In houses, this line is normally considered to extend from edge to edge of the diaphragm for which the shear wall(s) provides resistance. The line may be considered to occur at either edge of the wall or at the center line of the wall thickness.

Lines Acting Together — Two or more parallel lines of shear resistance so close to each other that the shear walls along the lines tend to distribute the total horizontal force applied amongst them as if they were located along a single line.

Loads

Seismic Load: Percentage of the vertical weight of a structure applied to the structure as a horizontal force in order to approximate statically the dynamic effects of earthquake motion. Seismic Loads are applied on a level-by-level basis (2nd floor, roof, etc.) with the total load at each level being proportional to the vertical weight of that level. As used in this report, the terms "load" or "total load" refer to the horizontal force to a line of resistance (or lines acting together) as well as to individual shear panels.

Unit Load: Equivalent seismic force generated by one square foot of a structure at a single "horizontal" level. Since walls also generate lateral forces, their effect is averaged into the Unit Load for each horizontal level of the structure, yielding Total Unit Load per square foot for that level.

Vertical Load: Portion of weight of a structure acting on an individual structural element, such as a wall.

Wind Load: Horizontal force applied to a structure as a result of wind blowing against it. Although acting on external elements only, wind load is apportioned level-by-level, with subsequent design of various structural elements then being similar to designs for seismic load.

Multiplying Factor — Multiplier of not less than 1.25 and not exceeding 2.0, used to increase the tributary width (hence the load) to interior shear walls at the first floor of two-story houses. The multiplier is used because of the relative stiffness of a small wood diaphragm compared to the shear walls, and as compensation for both variability bility in length and relative lengths of the exterior shear walls at each end of the diaphragm.

Nailing of Shear-Resisting Materials — Required fastening for finish materials is indicated in Tables 3.9 and 3.10. For the differing nail spacing at the edges and interior of sheets of finish material: Boundary Nailing — nailing at the periphery of the diaphragm; Edge Nailing — nailing to other edges of the sheets; Field Nailing — fastening to supports at the interior of sheets.

Overturning — Mechanical principle requiring a shear wall to attempt to rotate about one end of its base when subjected to horizontal load. Resistance to overturning is proportional to the vertical load on the wall and the base anchorage of the wall, if any.

Roof Level — Plane or planes created by the roof rafters, frequently not at a constant elevation. Bottom of the rafters, or where no rafter occurs, bottom of the sheathing along a given line. Normally the top of the top plate of a stud wall that extends to the

1.2 roof. Bottom chords of prefabricated roof trusses at the line of truss support only. Between points of truss support, bottom chords of roof trusses are considered as ceiling framing, not as part of the roof level. This definition is used primarily to differentiate between those interior walls that extend to the roof level and those that stop at the ceiling level. Exterior walls are always considered as extending to the roof level.

Seismic Force – Used synonymously with Seismic Load, see Loads.

Seismic Zones – Designations of geographical areas within which historical evidence has indicated that the intensity of earthquakes (and their effect upon structures) is approximately the same. Zone 3 designates areas of maximum expected intensity and requires that structures be designed for the greatest horizontal accelerations. Zone 2 uses one half the equivalent static force of Zone 3, and Zone 1 uses one fourth.

Shear

Shear-per-foot: Total shear at the boundary of a diaphragm, vertical or horizontal, divided by the length of the diaphragm. The methodology of this report requires shear-per-foot to be determined only for wood stud shear walls.

Total Shear: As used in this report, Total Shear is synonymous with the total load for a vertical or horizontal diaphragm at a line of resistance, lines acting together, or a single shear panel. Example: If 4000 lbs is the total load to a line of resistance consisting of four 5'-0" shear panels, Total Shear to the line is 4000 lbs and the Total Shear in any one panel is 4000/4 = 1000 lbs. Technically, Total Shear within a horizontal diaphragm is only that portion of the load developed to one side of the line of resistance. If the 4000-lb load to an interior line of resistance were divided with 2500 lbs coming from tributary area to the right of the line and 1500 lbs from tributary area to the left, the maximum Total Shear within the diaphragm would be 2500 lbs; Total Shear to the line would still be 4000 lbs.

Shear Transfer: Interconnection of materials, usually by mechanical means, for the purpose of transferring shear load from one material to another.

Shear Wall: Length of wall connected to a horizontal diaphragm to provide seismic resistance for the horizontal diaphragm and transfer its load to the ground or to the story below. Shear walls may be constructed of wood studs (developed as a vertical diaphragm), masonry, concrete or any other material capable of so functioning.

Shear Wall Height: Distance or average distance from the bottom of the sole plate to the top of the top plates of a shear panel, normally within a single story.

Shear Wall Length: Longitudinal dimension of a shear panel as viewed in plan. Overall or total length (as referred to on Calc Forms 8 and 9) is the total of the lengths of all shear panels along a line of resistance or lines acting together.

Shear Wall Width: Same as Shear Wall Length. Width is usually used when the shear panel is visualized in elevation; length is associated with plan view. Secondarily, width is used to describe the thickness of the wall as viewed in plan.

Sill Bolts – Anchor bolts installed in a foundation or foundation wall to connect a wood sill plate used as the base for wood frame construction.

1.2 **Sill Plate** – Wood member, connected to the foundation or a concrete or masonry wall, that serves as a base for wood frame construction. The sill plate may be a mudsill upon which first floor joists rest, the sole plate of a wood stud wall or the plate at the top of a one-story masonry wall to which the second floor joists are fastened.

Sole Plate – Horizontal wood member, forming the base of a wood stud wall unit, fastened to either the wood floor framing below or to the foundation.

"Solid" Wall – For purposes of considering overturning resistance, a wall or section of wall including one or more shear panels and one or more openings which, because it meets conditions set forth in Section 6.4A, may be treated as if the openings did not exist.

"Solid" Wall Length – Length of "solid" wall used for determining overturning resistance.

Split-Entry Construction – See Section 2.3C.

Split-Level Construction – See Section 2.3A1.

Strut – Device for collecting load from a diaphragm and transferring it, through tension or compression, to a shear wall. Strut action is usually provided in residences by the continuous top plates of wood stud walls or the reinforced lintels of masonry walls. Other struts are rarely required but, when used, usually consist of rafters or floor joists, or blocking between them, interconnected with a steel strap to provide tensile resistance. For the latter types of struts to be effective, adequate connections must be made to the top plates of the wall line containing the shear-resisting panel(s).

Tributary Area – Area of a roof or floor that contributes load to a line of resistance, a single line or lines acting together. All area, including overhangs, is considered, as are all loads developed within the area from ceilings, interior and exterior walls, etc., and special loads such as from masonry fireplaces.

Tributary Area (Length): Dimension parallel with the direction of load being considered. Unlike Diaphragm Length, this dimension represents the total dimension perpendicular to width of the area (including overhangs) contributing load.

Tributary Area (Width): For the direction of load being considered, dimension of the tributary area perpendicular to the direction of load (as viewed in plan). For exterior walls – generally one half the distance to the next adjacent parallel shear wall plus the full dimension of any overhang. For interior walls – one half the full distance between the next adjacent parallel shear walls to either side. Special cases can occur with houses of irregular plan, but the area determined should always represent the true area using the general rules herein. See also Multiplying Factor. (Detailed description of determining tributary widths in Section 3.3.)

Tributary Vertical Load: Vertical load supported by a single structural element. Vertical load to shear walls, for instance, may be made up of load from the roof, ceilings and second floor as well as the weight of the wall itself. (Detailed description of determining tributary vertical load to shear walls in Section 6.4B.)

Tri-Level Construction – See Section 2.3A1b.

Weight – As used in this report, solely the weight-per-square-foot of a material or structural system comprised of a group of materials. Structural components such as wall framing or floor joists are noted in terms of their weight-per-square-foot within the system rather than by actual weight. A 2 x 8 may weigh 2.5 lbs/ft but if it is 2'-0" o.c. its weight (as far as this report is concerned) is 1.25 psf.

SECTION 2: DETERMINATION OF SHEAR WALL LOCATIONS

2.1 One-Story Construction

A. Requirements for Shear Walls

1. Wall panels that extend continuously from the floor level to the roof level shall be considered as shear walls provided they have a minimum length of 4'-0" and meet the height-to-width ratio requirements for the shear-resisting material used. Interior walls stopping at the ceiling level shall not be considered as shear walls unless struts are provided.

   a. The height-to-width ratio of shear panels using any approved shear-resisting material other than plywood shall not exceed 2:1.

   b. The height-to-width ratio of shear panels using plywood sheathing shall not exceed 3.5:1.

   Exception: Garage walls adjacent to garage doors may be considered as shear walls when they are constructed as shown on Detail 45/4 provided the only seismic force transmitted to those walls is from the garage itself.

2. An approved shear-resisting material as specified in Section 6.3C shall extend the full height of the wall on at least one side of all walls used as shear walls.

3. Holes not wider than 2'-0" nor taller than 3'-0" may be placed in shear walls for the provision of utility boxes, medicine chests, etc. When the hole does not extend through the wall and does not interrupt the shear-resisting material, the hole may be ignored.

   When the hole occurs on the same side and, therefore, interrupts the shear-resisting material, the length of the wall minus the width of the hole shall be not less than 3'-0". For purposes of determining shear resistance, the effective length of the shear wall shall be the length of the wall minus the width of the hole. Holes shall not be placed within 16 inches of the vertical edges of the shear panel and shall not interrupt the sill plate, sole plate or the top plate of the wall. The height-to-width ratio of the wall adjacent to the opening shall be determined by dividing the height of the opening by the least width to either side of the opening. This ratio shall not exceed the requirements of Section 2.1A1.

B. Location of Shear Walls

1. Exterior Walls. Effective shear walls shall be located along all lines of exterior walls. Where no qualifying wall exists along an exterior wall, a cantilevered diaphragm having other shear wall support as defined by this Section must be provided. Each single line of exterior wall shall meet this requirement. Section 3.2 shall not be interpreted as applying to the required location of shear walls. Shear walls shall be placed at least every 25 feet along the length of each exterior line of resistance and, where design permits, be located at or near each exterior and interior corner.

2.1B 2. Interior Walls. All interior walls extending to the roof level that conform to Section 2.1A shall be designed as shear walls. Interior walls required to be considered as shear walls normally include but are not necessarily limited to:

a. Attached garage fire walls.

b. All walls enclosing spaces having cathedral ceilings or exposed roof framing.

c. Walls common to two adjacent spaces having different roof elevations, such as the interior wall common to the two levels of a split-level house.

Exception: Houses with exposed roof framing in several adjoining rooms, houses with flat roofs and houses with other similar conditions having many interior walls extending to the roof level need not have every interior wall designed as a shear wall provided the diaphragm ratio between any adjacent lines of resistance does not exceed 1.5:1. Determination of interior shear walls required for these conditions shall be in accordance with Section 2.2B2b. This exception does not apply to walls common to attached garages or carports or to walls having different types of roof sheathing on each side.

3. Cantilevers. Cantilevered diaphragms shall not cantilever more than half the distance of the back-span. Back-span shall be measured as the distance from the line of resistance from which the cantilever is measured to the farthest parallel line of resistance directly behind the cantilever. A diaphragm shall not be considered as cantilevered if a wall otherwise eligible as a shear wall occurs under the cantilever portion of the diaphragm.

2.2 Two-Story Construction

A. Requirements for Shear Walls

1. Second Story. The requirements stipulated in Section 2.1 shall apply to all shear walls extending from the second floor to the roof of two-story construction. The floor level referred to in Section 2.1A shall be considered to be the second floor for the purposes of this section.

Second-story shear walls shall be connected to the second floor diaphragm as indicated by Table 4.5, Nailing Schedule, for the appropriate Seismic Zone.

2. First Story. Shear walls extending from the first to the second floor of two-story construction shall meet the requirements of Section 2.1A with the Exception deleted.

Exception: Garage walls adjacent to garage doors may be used in lieu of shear walls when they are constructed as shown on Detail 46/4 subject to the following provisions:

a. The only seismic force transmitted to these walls is from the garage and the structure immediately above. The second floor may be cantilevered 4'-0" maximum beyond the garage door or beyond a side wall, however.

b. The detail shall be considered as providing resistance to rotation of the

TABLE 3.1

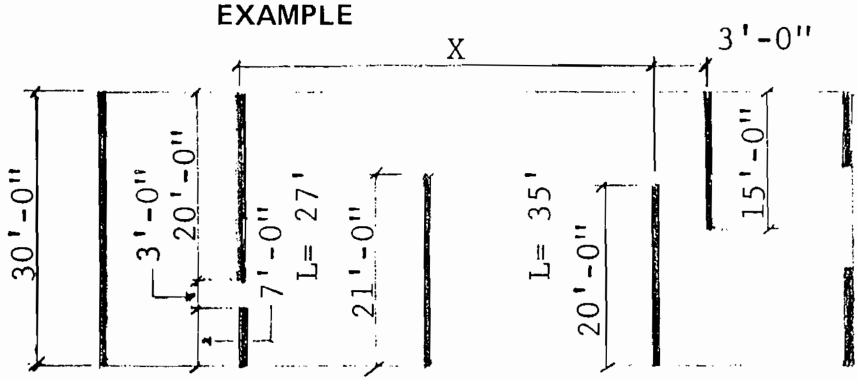
DETERMINATION OF REQUIRED USE OF INTERIOR WALLS AS SHEAR WALLS*

If the diaphragm ratio between the nearest lines of resistance on each side does not exceed:	Potential line(s) of resistance need not be considered if ratio of total effective wall length on the line(s) to shortest total effective length on line(s) to either adjacent side does not exceed:
1/2 : 1	**0.90**
3/4 : 1	**0.85**
1 : 1	**0.80**
1-1/2 : 1	**0.75**

* Must be used at 1st story of two-story houses (Section 2.2B2b).
May be used with one-story construction (Section 2.1B2).

EXAMPLE

DETERMINE IF 21'-0" LONG WALL MUST BE DESIGNED AS A SHEAR WALL



Shortest adjacent wall length = 27'-0"

$$\frac{21}{27} = 0.778$$

Diaphragm ratio $= \dfrac{X}{30}$

X = 45'+:	**Diaphragm ratio greater than 1-1/2:1. Wall must be used.**
X = 30' to 45':	**Diaphragm ratio greater than 1:1. 0.778 is greater than 0.75. Wall must be designed.**
X = 30' or less:	**0.778 is less than 0.80. Wall need not be considered.**

2.2A2b diaphragm and not as a shear wall per se. One or more interior walls shall be used as shear walls in conjunction with this detail with tributary areas to the interior and rear walls determined as specified in Section 3.3B4.

B. Location of Shear Walls

1. Second Floor. The requirements of Section 2.1B shall apply to the determination of locations of shear walls extending from the second floor to the roof of two-story houses.

2. First Floor. In all cases of two-story construction the second floor diaphragm shall have a diaphragm ratio not exceeding 1.5:1. Interior shear walls shall be provided as necessary to satisfy this requirement.

   The following additional requirements shall be met in locating shear walls at the first-story level of two-story construction.

   a. Exterior Walls. Shear walls shall be provided such that at least 20% of the length of each exterior wall is shear wall except where a cantilevered diaphragm is provided. Each single line of exterior wall as well as walls supporting a cantilevered diaphragm shall meet this requirement. Section 3.2 shall not be interpreted as applying to the required location of shear walls.

      Where design permits, shear walls shall be located at or near all exterior and interior corners.

   b. Interior Walls. Interior walls shall be designed as shear walls whenever the effective length of the wall or the total effective length along a line or lines of potential shear resistance (see Section 3.3B2) exceeds the ratio given in Table 3.1. The ratio shall be determined by dividing the total effective length of interior wall along a potential line or lines of shear resistance by the shortest total effective length of shear wall to either side of the wall(s) under consideration.

   c. Diaphragm Openings. A shear wall shall be provided under at least one of the longitudinal edges of openings in the second-floor diaphragm (for stairwells, etc.) when such opening decreases the effective length of the diaphragm by more than 50%.

3. Cantilevers. Cantilevered diaphragms shall not cantilever more than half the distance of the back-span. Back-span shall be measured as the distance from the line of resistance from which the cantilever is measured to the farthest parallel line of resistance directly behind the cantilever. A diaphragm shall not be considered as cantilevered if a wall otherwise eligible as a shear wall occurs under the "cantilever" portion of the diaphragm.

For two-story construction other than simple, conventional two-story construction, the location of shear walls shall be as required by Section 2.3.

2.3 Split-Level, Discontinuous Roof, Split-Entry and Combination One- and Two-Story Construction

A. Split-Level Construction

1. Definitions.

   a. Single-Story Split-Level: Single-story residence having two or more floor levels separated vertically by 2'-6" or more. Roof need not be discontinuous.

   Exception: Depressed floors within a room or other designs that do not require shear wall heights to vary by more than 2'-6" may be considered as standard one-story construction.

   b. Tri-Level: Combination one- and two-story residence with the floor level of the one-story portion at an elevation between the first and second floor elevations of the two-story portion.

2. Location of Shear Walls. Shear walls shall be located in each portion of split-level houses in accordance with the applicable sections: 2.1B for one-story portions, 2.2B for two-story portions. Each portion shall have shear walls so placed that each separate diaphragm receives the lateral support required by other sections of these requirements.

B. Discontinuous Roof Construction

1. Definition. Residence having two or more roofs separated vertically at the point they appear to adjoin (plan view), such that the roof sheathing is discontinuous.

2. Location of Shear Walls. Shear walls shall be located in houses with discontinuous roofs in accordance with the applicable sections: 2.1B for one-story portions, 2.2B for two-story portions. Each portion shall have shear walls so placed that each separate diaphragm receives the lateral support required by other sections of these requirements.

C. Split-Entry Construction

1. Definition. Two-story residence with entryway located at approximately mid-height between first and second floors.

2. Location of Shear Walls. Shear wall shall be provided between the first and second floors on at least one side of the entry. The sides of the entry are defined as those edges perpendicular to the exterior wall through which entrance is made.

D. Combination One-and Two-Story Construction

1. Definition. Partial one- and partial two-story house with first floor of each portion at same elevation. Tops of one-story and first-story walls at same elevation.

2. Location of Shear Walls. Combination one- and two-story houses shall either have a wall common to the two portions where they adjoin, or shall provide a line of shear resistance in the first story of the two-story portion such that the second floor diaphragm can cantilever to the line of joining. Such cantilevers shall be limited to 6' from line of resistance to line of joining. When an exterior wall of a combination house is in a single plane and the top plates are in a straight line (continuous), the shear wall for both portions may be located in either portion.

SECTION 3: TRIBUTARY AREAS

3.1 Guidelines

The general guidelines set forth in this section assume the following:

A. The floor plan used is the plan for the floor for which shear walls are to be designed (second floor plan for second floor shear walls, etc.).

B. Each direction of load shall be considered separately. Walls not parallel (or approximately parallel) to the direction of load under consideration shall not be included in interpreting these guidelines.

C. The width of the tributary area is the dimension perpendicular to the shear wall as viewed in plan. The length of the area is the dimension parallel to the line of shear resistance as viewed in plan.

3.2 Lines of Resistance Acting Together

A. Two lines of shear resistance 6′ or less apart (measured perpendicular to the lines in plan view) shall be assumed to be acting together.

B. Two lines of shear resistance more than 6′ but not more than 9′ apart (perpendicularly) may be assumed to be acting together provided the distance to the nearest parallel line of resistance to either side is at least 1-1/2 times the distance between them.

Exception: Where the diaphragm changes its length between the shear walls such that the shortest length of diaphragm is less than 2/3 the longest length, the walls shall not be considered as acting together.

C. Three or more lines of shear resistance with a distance between the outermost lines of 9′ or less shall be assumed to be acting together.

Exception: When the diaphragm changes its length (as defined in the Exception above) between such parallel lines of resistance, walls to either side of the change in length of the diaphragm shall not be considered as acting together.

D. When walls not required to be used as shear walls under Section 2.2B2b are nevertheless utilized, they shall meet the requirements of this section.

3.3 Calculating Tributary Widths and Lengths

Deflections of shear walls in the plane of the wall may be approximated as proportional to shear-per-foot. No formulation of this principle is provided, but judgmental decisions on the apportioning of load for cases not covered by these guidelines should take this principle into consideration.

A. One-Story Houses and Second Story of Two-Story Houses.

1. The distance between adjacent parallel shear walls shall be divided in half. All area to the left of the dividing line shall be considered as contributing load to the left shear wall. Similarly, all area to the right shall be considered as con-

tributing load to the right shear wall. Varying depth of diaphragm shall not be considered as having an effect on width of tributary area.

2. When lines of resistance are assumed to be acting together, each portion of tributary width shall be figured between the next adjacent line of resistance and the closest of the lines acting together. Area developed between the lines acting together shall be considered as contributing load to those lines.

3. Wings.

The line of resistance at a wing of a house that has an interior corner at one end and an exterior corner at the other need not be considered as a line of resistance for the main body of the house unless:

a. The total effective length of shear wall along the line is more than 3/4 as long as the total effective length of wall at the nearest parallel line of resistance for the main body of the house; or,

b. The total effective length of shear wall along the line exceeds 15'.

When exceptions a. and b. do not apply, tributary areas to such lines of resistance may be considered to be developed within the wing of the house contiguous to the line only.

4. Cantilevered Roof Diaphragms

All cantilevered diaphragm area shall be considered as tributary area to the outermost line of resistance adjacent to that area.

B. First Story of Two-Story Houses

1. Tributary width between lines of resistance shall be calculated as indicated in Section 3.3A1.

2. Lines of resistance assumed to be acting together shall be considered as set forth in Section 3.3A2.

Potential interior lines of resistance separated more than 3'-0" on center need not be considered as acting together in determining whether they need be designed as shear walls per Table 3.1.

3. The width tributary to an interior line of resistance next adjacent to a parallel exterior line shall be increased using the requirements shown in the following subsections. In interpreting these requirements, lines acting together shall be considered as a single line of resistance and unadjusted width shall be determined in accordance with Section 3.3A2. The only width to be adjusted in

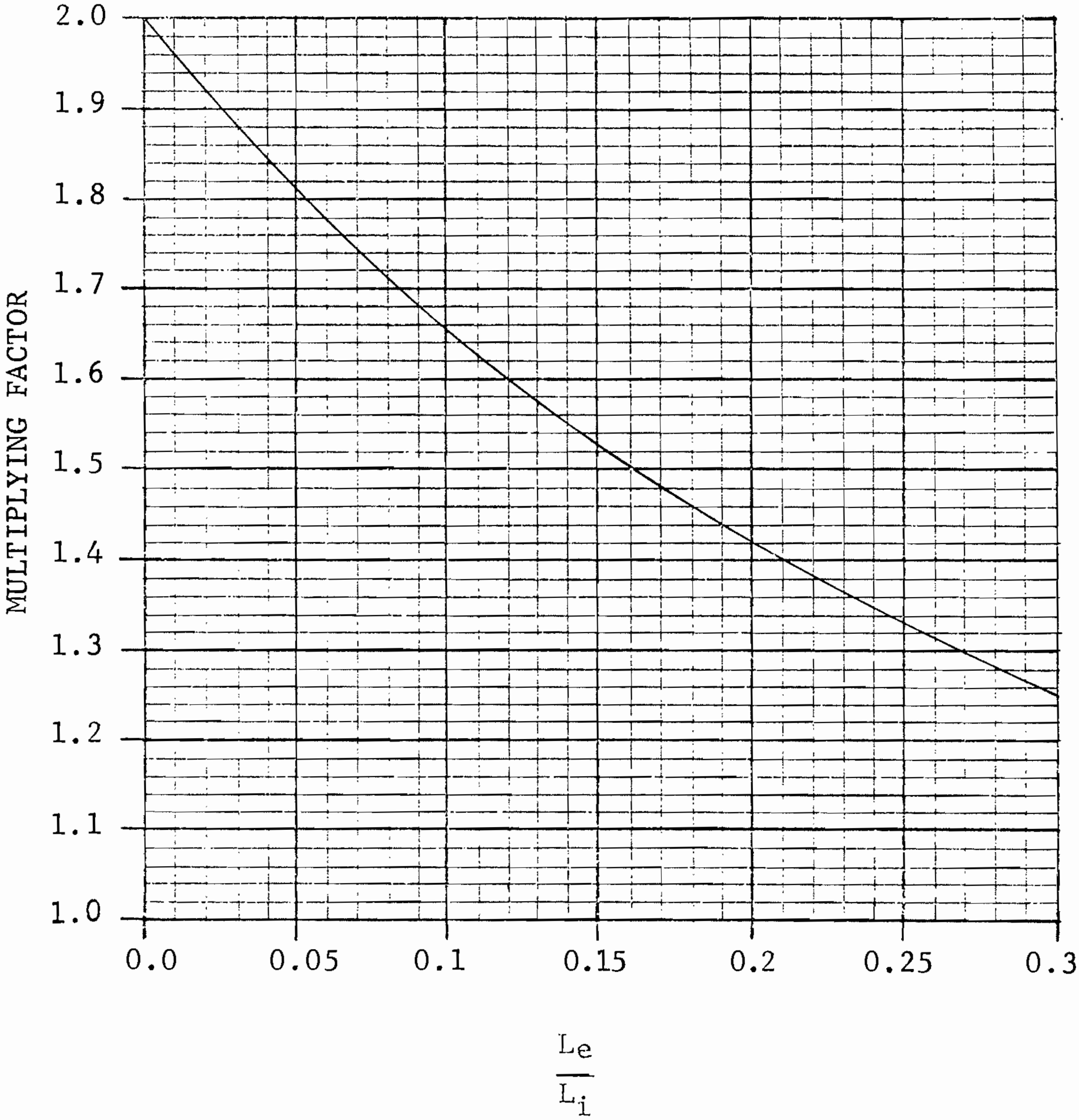


FIGURE 3.2 Multiplying Factor for Tributary Width Adjustment (Section 3.3B3).

3.3B3 the following subsections (3.3B3a through c) is that portion tributary to an interior wall and falling between an interior and exterior wall.

a. Where a single interior line of resistance is provided in addition to the lines of resistance at the parallel exterior walls, the ratio:

$$\frac{L_e}{L_i}$$ shall first be determined,

where

L_e = total effective length of shear wall at the parallel exterior wall having the shortest effective length of shear wall; and,

L_i = total effective length of interior shear wall.

When this ratio is greater than 0.30, the tributary width to the interior line of resistance shall be multiplied by 1.25. Tributary widths to exterior walls shall not be correspondingly decreased.

When the ratio is less than 0.30, Figure 3.2 shall be used to determine the multiplying factor required for adjusting the tributary width. See Figure 3.3 for example.

b. When only one interior line of resistance is provided but is made up of lines acting together, the total tributary width to the interior lines minus the distance between the lines acting together shall be multiplied as indicated in subsection a. above. See Figure 3.4 for example.

c. When more than one interior line of resistance (not acting together is provided, the ration L_e / L_i shall be determined for the first interior parallel line of resistance adjacent to each exterior line. L_e shall be the total effective length of shear wall along the exterior wall adjacent to the interior wall being considered. The 1.25 multiplying factor or the larger multiplying factor indicated in Figure 3.2 shall be applied to the width to the interior wall occurring between the interior and exterior wall only. See Figure 3.5 for example.

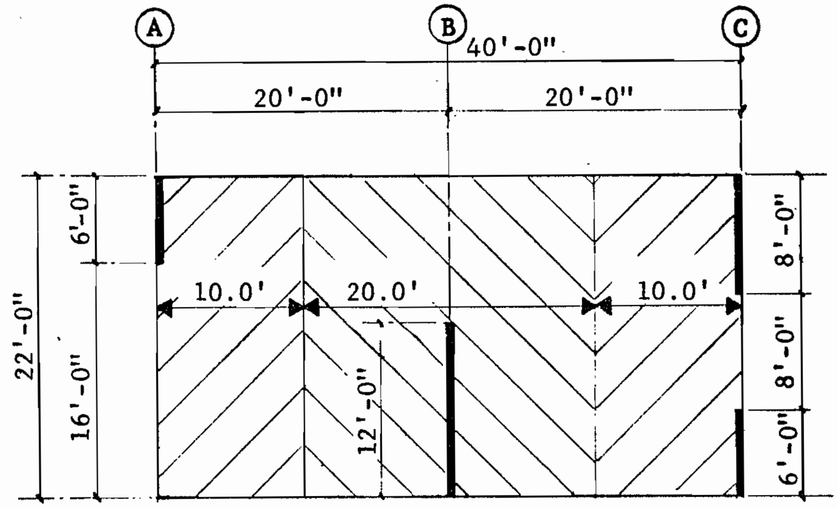
When properly utilized, the effect of the above requirements will be to create tributary widths for interior walls which overlap with the widths to the adjacent exterior walls.

4. When Detail 46/4 (Special Garage Front Wall Detail) is used, tributary widths shall be determined as follows:

a. When a single interior line of resistance (or lines acting together) is present, the width tributary to this line shall be determined by doubling the distance from the edge of the second floor diaphragm (above or beyond the line of the garage doors) to the rear wall of the garage.

Note: When the depth of the structure behind the garage is less than the depth of the garage, the effect of this provision will be to create a width used for design purposes greater than the depth of the structure.

b. When more than one interior line of resistance is present, tributary width to

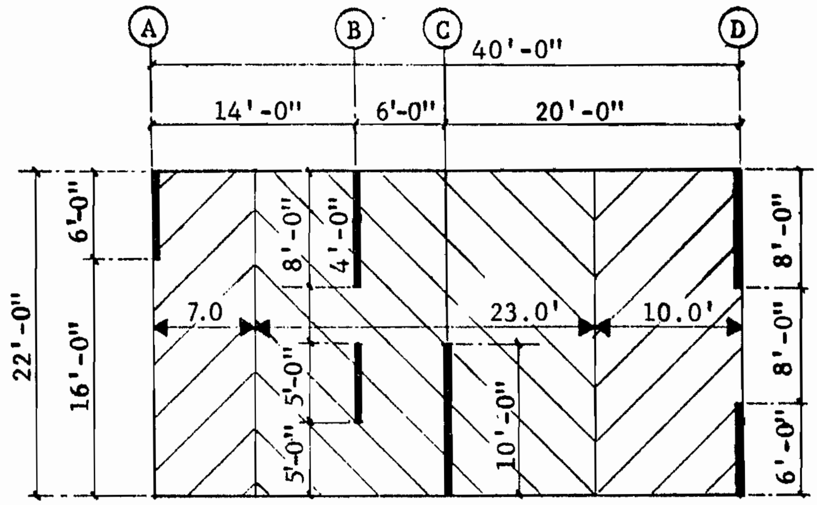


$$L_e = 6'\text{-}0'' \quad \frac{L_e}{L_i} = \frac{6.0}{12.0} = 0.50$$
$$L_i = 12'\text{-}0''$$

Multiplying Factor: 1.25

Adjusted Width, Wall B: 20.0′ x 1.25 25
Width at Walls A and C: 10.0′

FIGURE 3.3 Example Tributary Width Adjustment for Single Interior Line of Resistance — First Floor of Two-Story Construction

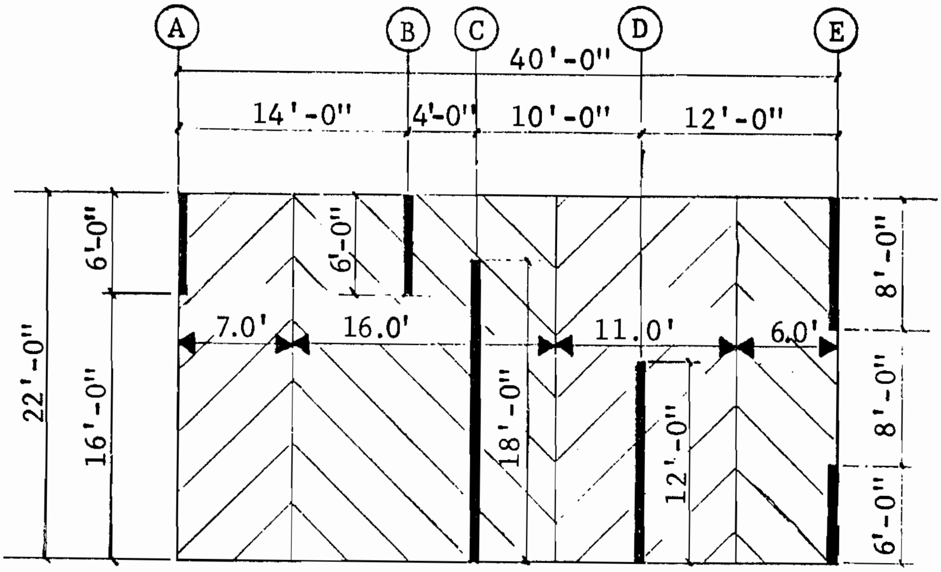


$$L_e = 6'\text{-}0''$$
$$L_i = 10.0 + 8.0 + 5.0 = 23.0 \quad \frac{L_e}{L_i} = 0.26$$

Multiplying Factor = 1.315

Adjusted Width, Walls B and C: (7.0 + 10.0) x 1.315 + 6.0 = 28.4′
Width at Line A = 7.0′ Width at Line D = 10.0′

FIGURE 3.4. Example Tributary Width Adjustment for Single Interior Line of Resistance (Lines Acting Together) — First Floor of Two-Story Construction.



At Left End: $L_e = 6.0$ $\quad L_i = 24.0$ $\quad \dfrac{L_e}{L_i} = \dfrac{6.0}{24.0} = 0.25$ Multiplying Factor = 1.33

Trib. Widths Line A: 7.0′

Lines B and C: 7.0 x 1.33 + 4.0 + 5.0 = 9.33 + 4.0 + 5.0 = 18.33′

At Right End: $L_e = 14.0$ $\quad L_i = 12.0$ $\quad \dfrac{L_e}{L_i} = \dfrac{14.0}{12.0} = 1.17$ Multiplying Factor = 1.25

Trib. Widths Line D: 5.0 + 6.0 x 1.25 = 5.0 + 7.5 = 12.5′ Line E: 6.0′

FIGURE 3.5. Example Tributary Width Adjustment for Two Interior Lines of Resistance — First Floor of Two-Story Construction.

3.3B4b the rear wall of the garage shall be determined utilizing the assumption that the second floor diaphragm over the garage is cantilevered from this wall. In addition to the normal method of determining tributary widths to other interior lines of resistance, the total width of the structure shall be assigned to all interior walls as if they were acting together. The load to each individual wall developed by this requirement shall be compared to the load developed by the normal requirements and each wall shall be designed for the greater of the loads.

c. The exterior wall at the opposite end of the structure shall have the width tributary to it determined in the manner specified in Section 3.3B1 without reference to the special requirements of this subsection.

5. Wings. The requirements of Section 3.3A3 shall apply.

6. Cantilevered Second Floor Diaphragm. The load developed by the tributary area of the cantilever shall be considered as contributing load to the line of resistance adjacent to the cantilever. When designs incorporate the Special Garage Front Wall Detail second floor diaphragms may not cantilever more than 4′-0″ beyond the line on which this detail is used.

SECTION 4: DETERMINATION OF SEISMIC LOADS

The Calc Forms discussed in this section are designed for use in calculating seismic forces to be resisted by the shear walls in a given residence. Calc Forms 1 through 5 facilitate a systematic evaluation of the actual weights and seismic weights of roofs, ceilings, floors and walls on a plan-area square footage basis. Calc Form 6 summarizes the plan area tributary to each line of resistance. Calc Form 7 provides a systematic procedure for multiplying the equivalent seismic weights by the tributary areas to determine the seismic forces to each line.

The seismic weights to be used on the Calc Forms vary with the seismicity in a given region. The seismic risk maps given in Figures 3.6 through 3.8 show hazard zoning for the United States. As noted earlier, these design recommendations apply only to Zone 2 and Zone 3 and, therefore, equivalent seismic weight tables referred to hereafter are developed only for Zone 2 and Zone 3.

The risk maps and applicable seismic weight tables are at the end of Section 4.

The blank Calc Forms provided may be removed and duplicated or the same information may be presented in some other format. When another format is used, however, all information required by the forms applicable to a given residential design shall be provided.

4.1 CALC FORM 1 – Summary of Roof, Ceiling and Wall Weights for Simple One-Story Residences

This form is to be used in designing one-story houses having a single interior wall finish, a single roofing type, a single ceiling finish and no more than two types of exterior wall finish other than veneer. When these conditions apply, Calc Form 1 may be used in place of Calc Forms 2 and 3 and the information called for shall be determined and set forth in accordance with the instructions for the appropriate parts of Calc Forms 2 and 3. (Treatment of Snow Loads is covered in Section 4.2C.)

4.2 Calc Form 2 – Summary of Roof, Ceiling and Floor Weights (Seismic Unit Loads) – For Other Than Simple One-Story Residences

This form shall be used for all residential designs (1) having a two-story portion, or (2) otherwise not conforming to the requirements for use of Calc Form 1.

Use the columns of boxes to provide the required information as follows:

A. Material. Identify all materials to be used for construction and finishing of the roof, ceiling and second floor.

B. Actual Weight. List actual weights in pounds per square foot. Weights of the most common materials and finishes are listed in Table 3.2.

CALC FORM 1

Job ______________________ Model ______________________

ONE-STORY RESIDENCE
ROOF, CEILING AND WALL WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight
Roofing			
Sheathing			
Framing			
TOTAL			
			Unit Load

CEILING	Material	Actual Weight	Seismic Weight
Framing			
Finish			
TOTAL			
			Unit Load

EXTERIOR WALLS	Material	Actual Weight	Material	Actual Weight
Framing				
Finish				
Finish				
TOTAL				

INTERIOR WALLS	Material	Actual Weight
Framing		
Finish		
Finish		
TOTAL		

"CEILING" UNIT LOAD

Ceiling __________

Exterior Walls __________

Interior Walls __________

TOTAL __________ psf

ROOF UNIT LOAD: __________ psf

VENEER LOAD: __________ lb/ft

CALC FORM 2

Job ______________________ Model ______________________

ROOF, CEILING AND FLOOR WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	Material	Actual Weight	Seismic Weight
Roofing						
Sheathing						
Framing						
TOTAL						

CEILING						
Framing						
Finish						
TOTAL						

2d FLOOR						
Flooring						
Sheathing						
Framing						
Ceiling						
TOTAL						

4.2B 1. Actual Weight of roof framing shall be considered to be not less than 2.0 pounds per square foot.

2. Actual Weight of ceiling framing shall be considered to be not less than 2.0 pounds per square foot.

C. Snow Loads

1. Vertical design snow loads of 32 psf and less may be ignored when determining seismic loads.

2. In areas requiring vertical load designs for snow loads in excess of 32 psf, 25% of the design snow load shall be added to the actual roof weight. The equivalent seismic unit load for this snow load may be determined from Table 3.3* Enter these loads on Calc Form 1 or 2 as applicable with the word "Snow" directly below the word "Framing" on the section of the form designated for Unit Roof Loads.

D. Seismic Weight. List the equivalent seismic weights as shown in Table 3.2. (Equivalent seismic weights for materials not tabulated may be determined from Table 3.3*)

Alternative: Find total of all actual weights for each category and determine seismic unit load corresponding to total actual weight from Table 3.3* Enter unit load under Total in seismic weight column.

4.3 Calc Form 3: Summary of Wall Weights and Seismic Unit Loads – For All But Simple One-Story residences

This form shall be used in conjunction with Calc Form 2. Use columns of the upper panels of boxes to provide the required information as follows:

A. Material. Designate finish material for each type of exterior and interior wall.

B. Actual Weight.

1. Actual Weight of wood stud wall framing shall be considered to be not less than 4.0 pounds per square foot. (Weights of many finishes are listed in Table 3.2.)

2. Tabulate all weights in pounds-per-square-foot of wall area. These panels do not reflect the effect of the wall weights on the structure. No equivalent seismic weights are therefore listed.

3. Total actual weight of wood stud walls with finishes shall be considered to be not less than 10.0 pounds per square foot.

*** Table 3.3 – Equivalent Seismic Weights – is made up individually for Zone 2 and Zone 3. The table numbering, 3.3, is the same for both zones. The designer must take care that he is using the table for the proper zone. This is also true for all other load tables (3.2 through 3.8).**
(Designers should remove Tables for redundant zone.)

4.3B 4. Do not include weight of veneer in wall weight.

C. Veneer Load. Determine veneer load as follows:

Compute actual weight per square foot of veneer by multiplying its thickness by its weight per inch of thickness. Brick or stone veneer shall be considered to weigh 10.0 pounds per square foot per inch of thickness. Weights of other veneers shall be ascertained from manufacturer or supplier.

Determine equivalent seismic weight from Table 3.3.

Calculation of veneer load shall be governed by height of veneer:

1. When veneer height is 2'-0" or less above finish first floor, its load may be neglected.

2. For one-story-high veneer, determine load to roof or second floor per foot of wall length by the formula:

$$w = \frac{vh}{2} x \frac{h}{H}$$

where: w = seismic load per foot of wall length due to veneer
v = equivalent seismic weight of veneer
h = height of veneer above floor line
H = floor-to-floor or floor-to-roof height

3. When veneer extends above the second floor line, calculate load from first to second floor as in (2), considering veneer below the second floor only, and calculate load to roof by the same method using weight and height of veneer from the second floor to the roof only. All load developed by veneer above the second floor that is not taken to the roof must be added to the second floor load. In no case shall equivalent seismic weight of any veneer above the second floor be neglected.

D. Unit Loads. Tabulate total unit loads contributed by the various elements of the structure in the bottom panel of boxes in Calc Form 3. Fill in the column headings as appropriate for the residence being designed.

1. Use the first column for total unit roof loads which normally should repeat the "Total" value calculated for Roof–Seismic on Calc Form 2. If more than one roof unit load is calculated, use succeeding columns to designate them.

2. Use remaining columns to designate "ceiling" unit loads, second floor unit loads, etc.

   a. List "Ceiling" unit load as shown on the appropriate Total line – Seismic of Calc Form 2.

   b. Obtain exterior unit wall loads from Table 3.6 or, when wall heights are other than 8'-0", from Calc Form 4 or 5 as applicable. When referring to Table 3.6 use actual weight of wall in pounds per square foot of wall area as determined in the upper portion of Calc Form 3 or 10 psf when wall weight is less.

CALC FORM 3

Job ______________________ Model ______________________

WALL WEIGHTS – SEISMIC UNIT LOAD SUMMARY
(In Pounds Per Square Foot)

EXTERIOR	Material	Actual Weight	Material	Actual Weight	Material	Actual Weight
Framing						
Exterior Finish						
Interior Finish						
TOTAL						

INTERIOR						
Framing						
Interior Finish						
Interior Finish						
TOTAL						

VENEER LOAD: __________ lb/ft

TOTAL UNIT LOADS	Roof					
Ceiling						
Exterior Walls						
Interior Walls						
TOTAL						

CALC FORM 4

Job ____________________ Model ____________________

UNIT WALL LOADS ADJUSTED FOR HEIGHT

Houses With Pitched Roof and Level Ceiling Below

	Adjustment	= W (psf)	Adjustment	= W (psf)	Adjustment	= W (psf)
ROOF						
$W_R = \frac{w_r h_r}{8}$	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
CEILING HEIGHT INTERIOR WALLS						
$W_2 = 0.72 \frac{w_2 h_r}{8}$	$0.72 \frac{\quad \times \quad}{8}$	=	$0.72 \frac{\quad \times \quad}{8}$	=	$0.72 \frac{\quad \times \quad}{8}$	=
$+ 0.28 \frac{w_2 h_2}{8}$	$0.28 \frac{\quad \times \quad}{8}$	=	$0.28 \frac{\quad \times \quad}{8}$	=	$0.28 \frac{\quad \times \quad}{8}$	=
$= W_2$						
EXTERIOR WALLS						
$E_r = \frac{e_r h_r}{8}$	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
$E_2 = E_r + \frac{e_2 h_2}{8}$	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=	$\frac{\quad \times \quad}{8}$	=
$= E_2$						

4.3D2 c. Obtain interior unit wall loads from Table 3.4, 3.5, 3.7 or 3.8 as appropriate or, when wall heights are other than 8'-0", from Calc Form 4 or 5. When referring to the tables use actual weight of interior partitions as set forth in the upper part of this Calc Form or 10 psf when actual weight is less.

d. To obtain the seismic load per square foot to be used at the second floor level, add total unit load from the second floor ceiling to the total unit load from the second floor.

4.4 Calc Form 4: Adjustment of Unit Wall Loads for Height in Houses With Pitched Roof and Level Ceiling Below

When wall heights are greater than 8'-0" and the house has a level ceiling and sloping roof, unit loads due to exterior and interior walls shall be adjusted using Calc Form **4.**

Exception: When one wall is shorter than the remainder of the walls (such as at the "common" wall of split-level houses) no adjustment need be made.

General considerations on wall height:

- Exterior wall heights that vary because of gabled roofs may be considered as the height of the exterior walls perpendicular to them for the purpose of all calculations on all the Calc Forms.
- When wall heights vary, a reasonable estimate of the average height may be used.

The adjustments to be made on Calc Form 4 are as follows:

A. Loads to the roof level from exterior and interior walls having a height other than 8'-0" shall be considered as ceiling loads but shall be adjusted before entering on the bottom part of Calc Form 3 by the formula:

$$W_r = \frac{w_r h_r}{8}$$

where W_r = adjusted unit load to be entered on Calc Form 3
w_r = unit load for 8'-0" high wall as obtained from Table 3.4 or 3.6
h_r = actual or actual "average" height of wall from bottom to top plates

B. Load to the second floor diaphragm from interior walls shall be adjusted for heights other than 8'-0" by the formula:

$$W_2 = 0.72 \frac{w_2 h_r}{8} + 0.28 \frac{w_2 h_2}{8}$$

where W_2 = adjusted unit load to be entered on Calc Form 3.
w_2 = unit load for 8'-0" high walls as obtained from Table 3.5
h_r = actual or actual "average" height of wall from second floor to roof
h_2 = actual height of walls from first to second floor

C. Load to the second floor diaphragm from exterior walls shall be adjusted for

4.4C heights other than 8'-0" by the formula:

$$E_2 = E_r + \frac{e_2\, h_2}{8}$$

where $E_r = \dfrac{e_r\, h_r}{8}$ and

where E_2 = adjusted exterior unit wall load to the second floor to be entered on Calc Form 3

E_r = adjusted exterior unit wall load to the "ceiling" level to be entered on Calc Form 3

e_r = unit load for 8'-0" high exterior wall as obtained from Table 3.6

e_2 = e_r when first and second story wall materials are the same and represents the portion of unit wall load from below the second floor

h_r = actual height of second floor walls from bottom to top plates

h_2 = actual height of first floor walls from bottom to top plates

4.5 Calc Form 5: Adjustment of Unit Wall Loads for Height — Houses with Flat Roofs or Sloping Ceilings

When wall heights are greater than 8'-0" and the house has a level roof with ceiling attached to the bottom of the rafters or when the walls extend to the sloping roof level, equivalent unit loads for exterior and/or interior walls shall be adjusted on Calc Form 5.

Exception: When one wall is shorter than the remainder of the walls (such as at the "common" wall of split-level houses) no adjustment need be made.

A. Use of Calc Form 5 shall be the same as for Calc Form 4 except that terms shall be defined as follows:

w_r = unit load for 8'-0" high wall as obtained from Table 3.6 or 3.7

w_2 = unit load for 8'-0" high interior wall as obtained from Table 3.8 (unit load from Table 3.7 is not to be added)

B. Load to the second floor diaphragm shall be adjusted for heights other than 8'-0" by the formula:

$$W_2 = \frac{w_r\, h_r}{8} + \frac{w_2\, h_2}{8}$$

4.6 Calc Form 6: Tributary Areas

Calc Form 6 shall be used as follows to calculate the tributary area for all shear walls being designed:

A. Enter the line of shear resistance under consideration in the first column. When lines of shear resistance are considered as acting together, enter both lines together.

B. Enter length and width of each area tributary to the line of resistance in the second column. When a multiplying factor is applicable (3.3B3) enter it also in the second column.

<u>CALC FORM 5</u>

Job ______________________ Model ______________________

UNIT WALL LOADS ADJUSTED FOR HEIGHT
Houses With Flat Roofs or Sloping Ceilings

	Adjustment	= W (psf)	Adjustment	= W (psf)	Adjustment	= W (psf)
ROOF						
$W_R = \frac{w_r h_r}{8}$	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=
FULL HEIGHT INTERIOR WALLS						
$W_2 = \frac{w_r h_r}{8}$	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=
$+ \frac{w_2 h_2}{8}$	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=
$= W_2$						
EXTERIOR WALLS						
$E_r = \frac{e_r h_r}{8}$	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=
$E_2 = E_r + \frac{e_2 h_2}{8}$	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=	$\frac{\quad x}{8}$	=
$= E_2$						

CALC FORM 6

Job ______________________ Model ______________________

TRIBUTARY AREAS

	ROOF AREA		CEILING AREA	
LINE	Length x Width	Area	Length x Width	Area

4.6 C. Enter in the third column the area obtained by multiplying length by width, or the area obtained by adding areas when more than one length and width are represented.

4.7 Calc Form 7: Seismic Loads

Calc Form 7 shall be used for the calculation of total seismic load to each line of shear resistance. Provide the required information in the columns of boxes as follows:

A. Line. List each line of resistance as shown on Calc Form 6 together with any special loads affecting the line such as masonry fireplace chimneys, veneer, garages, porches, etc., such that enough space is assured to make the ensuing calculations.

B. Roof Area. Enter roof area as shown on Calc Form 6 near the top of the box.

C. Unit Roof Load. Enter the seismic unit load from the roof alone as it is shown on either Calc Form 1 or 3.

D. Other Load. Enter calculations necessary for determination of other loads to be added to the roof load (such as masonry fireplaces, veneer, etc.) near the center of the appropriate box.

E. Total Roof Load. When load other than the unit roof load contributes to the total roof load, enter the product of multiplying roof area by unit roof load near the top of the box and enter the results of calculations from the previous column or other such loads directly below. Add the figures entered in the box for total roof load.

F. Ceiling Area. Enter ceiling area as shown on Calc Form 6 near the center of the box.

G. Unit Ceiling Load. Enter the seismic unit load from the "ceiling" as shown on either Calc Form 1 or 3. This load is the total shown and includes the effect of exterior and interior walls.

H. Total Load. If only unit roof load contributes to total roof load, multiply roof area by unit roof load and enter the product near the top of the box. If other load contributes, enter the total shown in the column headed Total Roof Load.

Enter the product of multiplying ceiling area by unit "ceiling" load at the center of the box. Total shall be obtained by adding all figures entered in this column for each line of resistance.

CALC FORM 7

Job ______________________ Model ______________________

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)

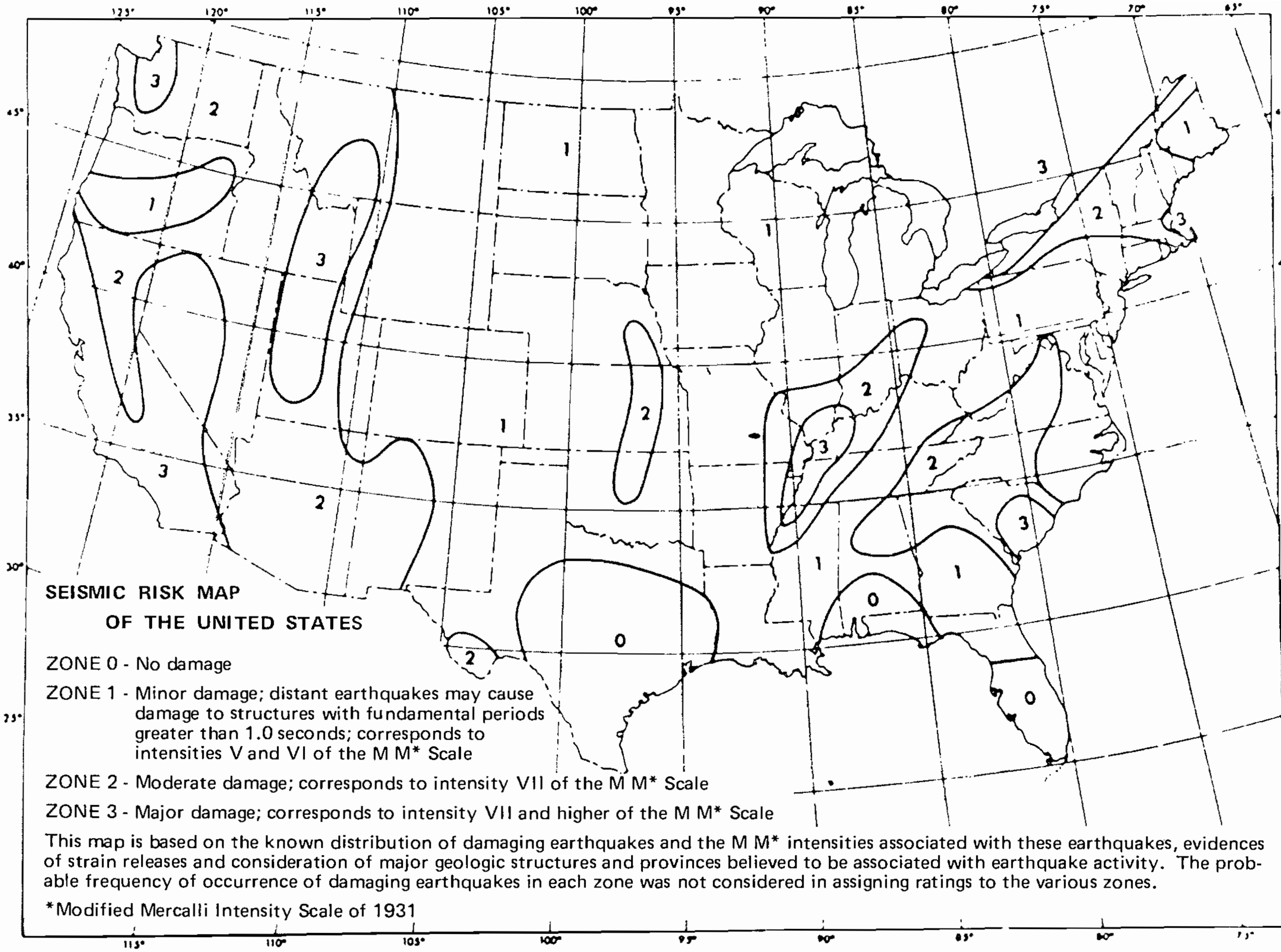


FIGURE 3.6. **Seismic Risk Map of the Contiguous United States, courtesy Uniform Building Code by International Conference of Building Officials.**

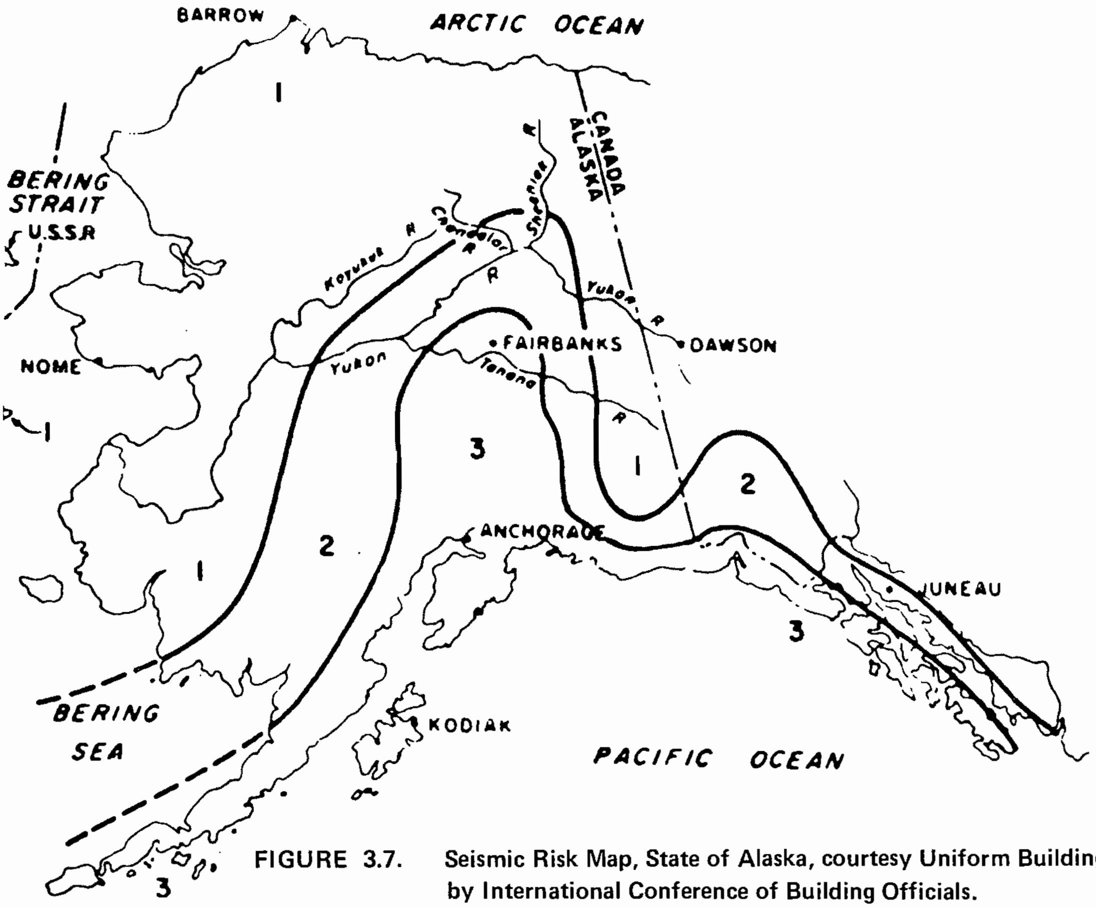


FIGURE 3.7. Seismic Risk Map, State of Alaska, courtesy Uniform Building Code by International Conference of Building Officials.

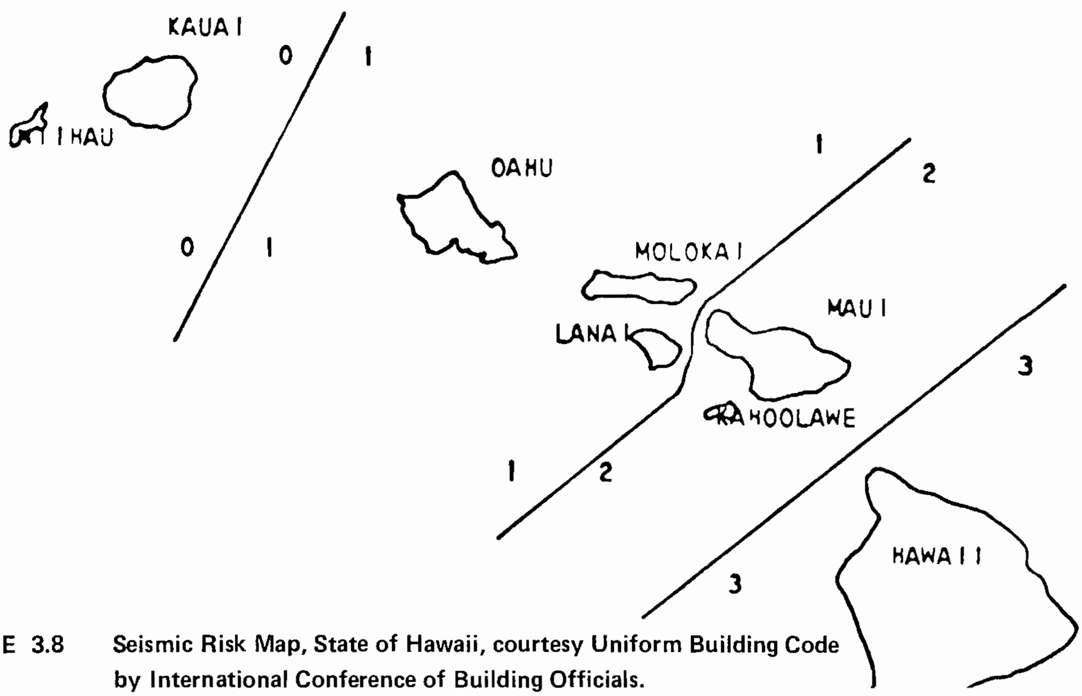


FIGURE 3.8 Seismic Risk Map, State of Hawaii, courtesy Uniform Building Code by International Conference of Building Officials.

TABLE 3.2 WEIGHTS OF MATERIALS AND EQUIVALENT SEISMIC WEIGHTS

Horizontal Framing (Roofs and Floors)

SUB-GROUP	MATERIAL	ASSUMED MAXIMUM WEIGHT (psf)	EQUIVALENT SEISMIC WEIGHT (psf)
Roofing(1)	Asphalt shingle, composition, Wood shingle or Wood shake	3.0	0.200
	Heavy shake	4.0	0.267
	Built up with gravel	6.0	0.400
	Decorative rock	6.0	0.400
	Spanish tile	14.0	0.933
	Concrete tile	16.0	1.067
Roof and floor Sheathing	Plywood to 3/4″	2.0	0.133
	Plywood to 1-1/8″	3.0	0.200
Roof Sheathing	1 x solid sheathing	2.0	0.133
	1 x spaced sheathing	1.5	0.100
	2 x solid sheathing	4.5	0.300
Underlayment	5/16″ particleboard	1.5	0.100
Framing	Prefabricated roof trusses, standard roof framing, standard ceiling framing or exposed beams	2.0	0.133
	2nd floor		
	To 2 x 12 @ 16	4.0	0.267
	To 2 x 14 @ 12	5.0	0.333
Flooring	Hardwood	2.0	0.133
	Lightweight concrete fill (per inch thickness)	9.0	0.600
	Carpet, linoleum, etc.	1.0	0.067
Ceiling Finish	1/2″ gypsum board (drywall)	2.0	0.133
	1/2″ gypsum board with spray-on acoustic finish	3.0	0.200
	5/8″ gypsum board (drywall)	2.5	0.167
	5/8″ gypsum board with spray-on acoustic finish	3.5	0.233
	3/8″ gypsum lath and 1/2″ plaster	8.0	0.533
Masonry fireplace chimneys	Reaction at second floor and/or roof		300.00

(1) Roofing materials vary considerably in weight. Actual weight of specific roof can be used with equivalent seismic weight determined from Table 3.3.

TABLE 3.2 Continued

TABLE 3.2 – ZONE 2 (Continued)

WEIGHTS OF MATERIALS AND EQUIVALENT SEISMIC WEIGHTS

Vertical Framing(2)

SUB-GROUP	MATERIAL	ASSUMED MAXIMUM WEIGHT (psf)	EQUIVALENT SEISMIC WEIGHT (psf)
Wood Framing(3)	2 x 4 or 2 x 6 or double 2 x 4	4.0	
Finishes(4)	3/8″ or 1/2″ plywood, 1 x sheathing, gypsum board, fiberboard, hardboard (one thickness, one side)	2.0	
	Stucco or gypsum lath and plaster	8.0	
	Stucco or plaster on face of block	5.0*	
	Furring inside block	1.0*	

*Add to block weight

Concrete Block In Pounds Per Square Foot

		LIGHTWEIGHT AGGREGATE Wall Thickness			SAND–GRAVEL AGGREGATE Wall Thickness		
		6″	8″	12″	6″	8″	12″
Solid grouted wall		56	77	118	68	92	140
Vertical cores grouted at:	16″ o.c.	46	60	90	58	75	111
	24″ o.c.	42	53	79	53	68	99
	32″ o.c.	40	50	73	51	65	93
	40″ o.c.	38	47	70	50	62	89
	48″ o.c.	37	46	68	49	61	87
	96″ o.c.	35	43	61	47	58	80

Masonry Veneer	Per inch of thickness	10.0 psf
Brick Walls	Per inch of thickness	10.0 psf

(2) No equivalent seismic weight is given for vertical framing. Determine actual weights (per square foot) of walls and refer to appropriate wall load tables for seismic weight per square foot of floor area.

(3) No wood stud wall shall be assumed to weight less than 10 psf total.

(4) All finishes given structural values are listed herein. Other finishes are too numerous to list. Weight per square foot should be determined and added to wall weight.

ZONE 3

TABLE 3.2 WEIGHTS OF MATERIALS AND EQUIVALENT SEISMIC WEIGHTS

Horizontal Framing (Roofs and Floors)

SUB-GROUP	MATERIAL	ASSUMED MAXIMUM WEIGHT (psf)	EQUIVALENT SEISMIC WEIGHT (psf)
Roofing[1]	Asphalt shingle, composition, Wood shingle or Wood shake	3.0	0.400
	Heavy shake	4.0	0.533
	Built up with gravel	6.0	0.800
	Decorative rock	6.0	0.800
	Spanish tile	14.0	1.867
	Concrete tile	16.0	2.133
Roof and floor Sheathing	Plywood to 3/4"	2.0	0.267
	Plywood to 1-1/8"	3.0	0.400
Roof Sheathing	1 x solid sheathing	2.0	0.267
	1 x spaced sheathing	1.5	0.200
	2 x solid sheathing	4.5	0.600
Underlayment	5/16" particleboard	1.5	0.200
Framing	Prefabricated roof trusses, standard roof framing, standard ceiling framing or exposed beams	2.0	0.267
	2nd floor		
	To 2 x 12 @ 16	4.0	0.533
	To 2 x 14 @ 12	5.0	0.667
Flooring	Hardwood	2.0	0.267
	Lightweight concrete fill (per inch thickness)	9.0	1.200
	Carpet, linoleum, etc.	1.0	0.133
Ceiling Finish	1/2" gypsum board (drywall)	2.0	0.267
	1/2" gypsum board with spray-on acoustic finish	3.0	0.400
	5/8" gypsum board (drywall)	2.5	0.333
	5/8" gypsum board with spray-on acoustic finish	3.5	0.467
	3/8" gypsum lath and 1/2" plaster	8.0	1.067
Masonry fire-place chimneys	Reaction at second floor and/or roof		600.00

(1) Roofing materials vary considerably in weight. Actual weight of specific roof can be used with equivalent seismic weight determined from Table 3.3.

TABLE 3.2 Continued

TABLE 3.2 – ZONE 3 (Continued)

WEIGHTS OF MATERIALS AND EQUIVALENT SEISMIC WEIGHTS

Vertical Framing[2]		ASSUMED MAXIMUM WEIGHT (psf)	EQUIVALENT SEISMIC WEIGHT (psf)
SUB-GROUP	MATERIAL		
Wood Framing[3]	2 x 4 or 2 x 6 or double 2 x 4	4.0	
Finishes[4]	3/8″ or 1/2″ plywood, 1 x sheathing, gypsum board, fiberboard, hardboard (one thickness, one side)	2.0	
	Stucco or gypsum lath and plaster	8.0	
	Stucco or plaster on face of block	5.0*	
	Furring inside block	1.0*	

*Add to block weight

Concrete Block In Pounds Per Square Foot

		LIGHTWEIGHT AGGREGATE Wall Thickness			SAND–GRAVEL AGGREGATE Wall Thickness		
		6″	8″	12″	6″	8″	12″
Solid grouted wall		56	77	118	68	92	140
Vertical cores grouted at:	16″ o.c.	46	60	90	58	75	111
	24″ o.c.	42	53	79	53	68	99
	32″ o.c.	40	50	73	51	65	93
	40″ o.c.	38	47	70	50	62	89
	48″ o.c.	37	46	68	49	61	87
	96″ o.c.	35	43	61	47	58	80

Masonry Veneer	Per inch of thickness	10.0 psf
Brick Walls	Per inch of thickness	10.0 psf

(2) No equivalent seismic weight is given for vertical framing. Determine actual weights (per square foot) of walls and refer to appropriate wall load tables for seismic weight per square foot of floor area.

(3) No wood stud wall shall be assumed to weight less than 10 psf total.

(4) All finishes given structural values are listed herein. Other finishes are too numerous to list. Weight per square foot should be determined and added to wall weight.

ZONE 2

TABLE 3.3 EQUIVALENT SEISMIC WEIGHTS

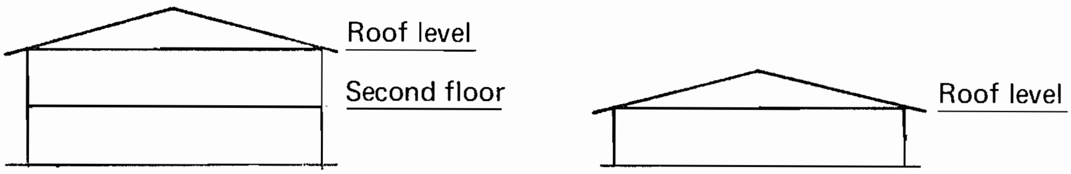
ACTUAL WEIGHT	SEISMIC WEIGHT	ACTUAL WEIGHT	SEISMIC WEIGHT	ACTUAL WEIGHT	SEISMIC WEIGHT
0.10	0.007	1.0	0.067	21.0	1.400
0.20	0.013	2.0	0.133	22.0	1.467
0.30	0.020	3.0	0.200	23.0	1.533
0.40	0.027	4.0	0.267	24.0	1.600
0.50	0.033	5.0	0.333	25.0	1.667
0.60	0.040	6.0	0.400	26.0	1.733
0.70	0.047	7.0	0.467	27.0	1.800
0.80	0.053	8.0	0.533	28.0	1.867
0.90	0.060	9.0	0.600	29.0	1.933
		10.0	0.667	30.0	2.000
		11.0	0.733	31.0	2.067
		12.0	0.800	32.0	2.133
		13.0	0.867	33.0	2.200
		14.0	0.933	34.0	2.267
		15.0	1.000	35.0	2.333
		16.0	1.067	36.0	2.400
		17.0	1.133	37.0	2.467
		18.0	1.200	38.0	2.533
		19.0	1.267	39.0	2.600
		20.0	1.333	40.0	2.667

ZONE 3

TABLE 3.3 EQUIVALENT SEISMIC WEIGHTS

ACTUAL WEIGHT	SEISMIC WEIGHT	ACTUAL WEIGHT	SEISMIC WEIGHT	ACTUAL WEIGHT	SEISMIC WEIGHT
0.10	0.013	1.0	0.133	21.0	2.800
0.20	0.027	2.0	0.267	22.0	2.933
0.30	0.040	3.0	0.400	23.0	3.067
0.40	0.053	4.0	0.533	24.0	3.200
0.50	0.067	5.0	0.667	25.0	3.333
0.60	0.080	6.0	0.800	26.0	3.467
0.70	0.093	7.0	0.933	27.0	3.600
0.80	0.107	8.0	1.067	28.0	3.733
0.90	0.120	9.0	1.200	29.0	3.867
		10.0	1.333	30.0	4.000
		11.0	1.467	31.0	4.133
		12.0	1.600	32.0	4.267
		13.0	1.733	33.0	4.400
		14.0	1.867	34.0	4.533
		15.0	2.000	35.0	4.667
		16.0	2.133	36.0	4.800
		17.0	2.267	37.0	4.933
		18.0	2.400	38.0	5.067
		19.0	2.533	39.0	5.200
		20.0	2.667	40.0	5.333

ZONE 2



WALL LOADS EXPRESSED
AS EQUIVALENT SEISMIC UNIT LOADS
FOR HOUSES WITH PITCHED ROOF AND CEILING BELOW[1]
(in psf of tributary area)

TABLE 3.4* UNIT LOAD FROM INTERIOR WALLS TO ROOF LEVEL

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.125	0.138	0.150	0.163	0.175	0.188
Wall weight	16	17	18	19	20	
Equivalent seismic load	0.200	0.213	0.225	0.238	0.250	

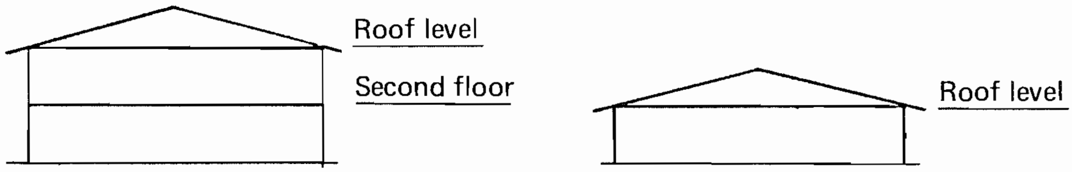
TABLE 3.5* UNIT LOAD FROM INTERIOR WALLS TO 2nd FLOOR DIAPHRAGM

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.750	0.825	0.900	0.975	1.050	1.125
Wall weight	16	17	18	19	20	
Equivalent seismic load	1.200	1.275	1.350	1.425	1.500	

1. Loads shown are for walls 8'-0" high. For higher walls multiply by ratio of heights.

***See footnote below Table 3.8**

ZONE 3



WALL LOADS EXPRESSED
AS EQUIVALENT SEISMIC UNIT LOADS
FOR HOUSES WITH PITCHED ROOF AND CEILING BELOW[1]
(in psf of tributary area)

TABLE 3.4* UNIT LOAD FROM INTERIOR WALLS TO ROOF LEVEL

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.250	0.275	0.300	0.325	0.350	0.375
Wall weight	16	17	18	19	20	
Equivalent seismic load	0.400	0.425	0.450	0.475	0.500	

TABLE 3.5* UNIT LOAD FROM INTERIOR WALLS TO 2nd FLOOR DIAPHRAGM

Wall weight	10	11	12	13	14	15
Equivalent seismic load	1.500	1.650	1.800	1.950	2.100	2.250
Wall weight	16	17	18	19	20	
Equivalent seismic load	2.400	2.550	2.700	2.850	3.000	

1. Loads shown are for walls 8'-0" high. For higher walls multiply by ratio of heights.

***See footnote below Table 3.8.**

ZONE 2

TABLE 3.6* EQUIVALENT SEISMIC UNIT LOADS – EXTERIOR WALLS – ONE STORY AND TOP FLOOR OF TWO-STORY[1] [2]
(in psf of tributary area)

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.375	0.413	0.450	0.488	0.525	0.563
Wall weight		16	17	18	19	20
Equivalent seismic load		0.600	0.638	0.675	0.713	0.750
Wall weight		21	22	23	24	25
Equivalent seismic load		0.788	0.825	0.863	0.900	0.938
Wall weight		26	27	28	29	30
Equivalent seismic load		0.975	1.103	1.050	1.088	1.125
Wall weight		31	32	33	34	35
Equivalent seismic load		1.163	1.200	1.238	1.275	1.313
Wall weight		36	37	38	39	40
Equivalent seismic load		1.350	1.388	1.425	1.463	1.500

1. At second floor, load shown shall be doubled except when adjustments are required for wall heights; masonry wall below, wood above, etc. Loads shown are those for a single 8'-0" wall height.
2. For walls other than 8'-0" high, multiply by ratio of heights.

***See footnote below Table 3.8**

ZONE 3

TABLE 3.6* EQUIVALENT SEISMIC UNIT LOADS – EXTERIOR WALLS – ONE STORY AND TOP FLOOR OF TWO-STORY[1] [2]

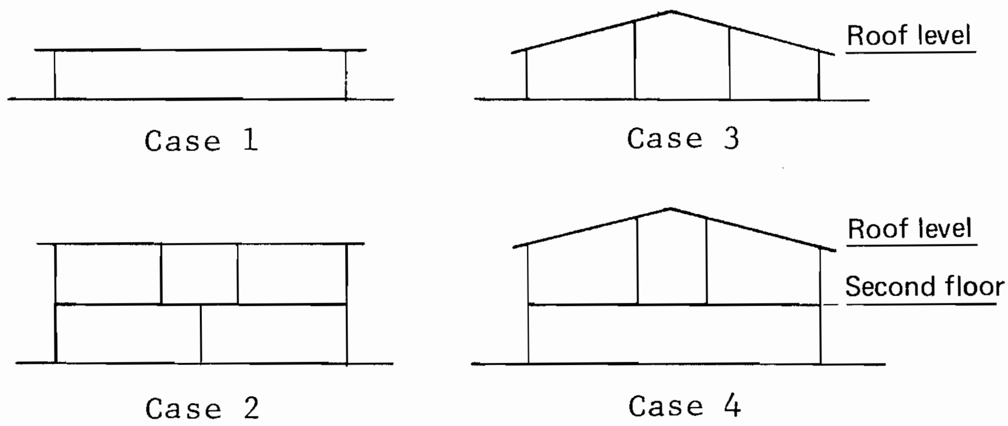
(in psf of tributary area)

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.750	0.825	0.900	0.975	1.050	1.125
Wall weight		16	17	18	19	20
Equivalent seismic load		1.200	1.275	1.350	1.425	1.500
Wall weight		21	22	23	24	25
Equivalent seismic load		1.575	1.650	1.725	1.800	1.875
Wall weight		26	27	28	29	30
Equivalent seismic load		1.950	2.025	2.100	2.175	2.250
Wall weight		31	32	33	34	35
Equivalent seismic load		2.325	2.400	2.475	2.550	2.675
Wall weight		36	37	38	39	40
Equivalent seismic load		2.700	2.775	2.850	2.925	3.000

1. At second floor, load shown shall be doubled except when adjustments are required for wall heights; masonry wall below, wood above, etc. Loads shown are those for a single 8'-0" wall height.
2. For walls other than 8'-0" high, multiply by ratio of heights.

*See footnote below Table 3.8

ZONE 2



WALL LOADS EXPRESSED
AS EQUIVALENT SEISMIC UNIT LOADS
FOR HOUSES WITH FLAT ROOFS AND HOUSES WITH SLOPING CEILINGS[1]
(in psf of tributary area)

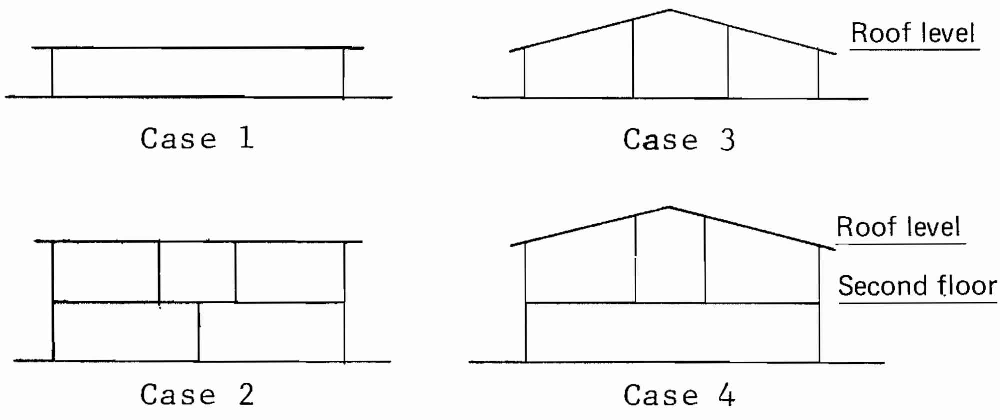
TABLE 3.7* UNIT LOAD FROM INTERIOR WALLS TO ROOF LEVEL DIAPHRAGM

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.333	0.367	0.400	0.433	0.467	0.500
Wall weight	16	17	18	19	20	
Equivalent seismic load	0.533	0.567	0.600	0.633	0.667	

1. Loads shown are for walls 8′-0″ high. For higher walls multiply by ratio of heights. Where interior wall height varies, such as Cases 3 and 4, estimate average interior wall height.

*See footnote below Table 3.8.

ZONE 3



WALL LOADS EXPRESSED
AS EQUIVALENT SEISMIC UNIT LOADS
FOR HOUSES WITH FLAT ROOFS AND HOUSES WITH SLOPING CEILINGS[1]
(in psf of tributary area)

TABLE 3.7* UNIT LOAD FROM INTERIOR WALLS TO ROOF LEVEL DIAPHRAGM

Wall weight	10	11	12	13	14	15
Equivalent seismic load	0.667	0.733	0.800	0.867	0.933	1.000
Wall weight	16	17	18	19	20	
Equivalent seismic load	1.067	1.133	1.200	1.267	1.333	

1. Loads shown are for walls 8'-0" high. For higher walls multiply by ratio of heights. Where interior wall height varies, such as Cases 3 and 4, estimate average interior wall height.

*See footnote below Table 3.8.

ZONE 2

TABLE 3.8* UNIT LOAD FROM INTERIOR WALLS TO SECOND-FLOOR DIAPHRAGM[1,2]

Wall weight	10	11	12	13	14	15
Equivalent seismic load	.210	.231	.252	.273	.294	.315
Wall weight	16	17	18	19	20	
Equivalent seismic load	.336	.357	.378	.399	.420	

1. Load shown in this table is for interior walls below second floor only. Add load (adjusted for height where necessary) from Table 3.7 to obtain total unit load to second floor diaphragm.

2. Loads shown are for walls 8'-0" high. For higher walls multiply by ratio of heights.

*NOTE:

Unit loads shown on Tables 3.4 through 3.8 express the effect of 8'-0" high walls on seismic loading in terms of equivalent load per square foot of house area for an "average" house. The loads shown are therefore approximate only and may be low for homes with many small rooms. For description of the use of these Tables see Section 4.3D.

ZONE 3

TABLE 3.8* UNIT LOAD FROM INTERIOR WALLS TO SECOND-FLOOR DIAPHRAGM[1,2]

Wall weight	10	11	12	13	14	15
Equivalent seismic load	.420	.462	.504	.546	.588	.630
Wall weight	16	17	18	19	20	
Equivalent seismic load	.672	.714	.756	.798	.840	

1. Load shown in this table is for interior walls below second floor only. Add load (adjusted for height where necessary) from Table 3.7 to obtain total unit load to second floor diaphragm.

2. Loads shown are for walls 8'-0" high. For higher walls multiply by ratio of heights.

*NOTE:
Unit loads shown on Tables 3.4 through 3.8 express the effect of 8'-0" high walls on seismic loading in terms of equivalent load per square foot of house area for an "average" house. The loads shown are therefore approximate only and may be low for homes with many small rooms. For description of the use of these Tables see Section 4.3D.

SECTION 5: WIND LOADS

5.1 Applicability

This section must not be considered as setting forth requirements for wind load design. Methods of calculation are presented solely to allow comparison of wind loads to shear walls with seismic loads to shear walls for locales where wind load designs are required. The wind loads determined in this section are only loads required to allow shear wall design. They do not include provisions for uplift, nor for design of individual members of a structure, nor do they cover other considerations pertinent to a full wind load design.

5.2 Determination of Wind Loads

Use wind loads as set forth in the MAP (shown in Figures 3.10 and 3.11) or as determined by local requirements.

A. One-Story or Second Floor of Two-Story

1. When determining the height of the area to which wind load should be applied, use one-half the height of the wall for one-story houses or one-half the height of the second floor wall for two-story houses plus, in either case, all roof height projected on a horizontal plane such as is shown on an exterior elevation.

2. Where height varies, use the average height for each successive tributary width.

3. Use the same width to each line of resistance as is used in determining seismic load.

4. Determine wind load by multiplying height by tributary width by the required wind load per square foot.

B. First Floor of Two-Story

1. In determining loads to first floor shear walls in two-story houses, use the total height of the structure minus one-half the first floor wall height as the total height of each successive tributary width.

2. Determine tributary widths as is done for seismic loads except that no multiplying factor (per 3.3B3) need be applied to interior shear walls.

3. Where height varies, use the average height for each successive tributary width.

4. Determine wind load by multiplying height by tributary width by the required wind load per square foot.

C. Overhangs

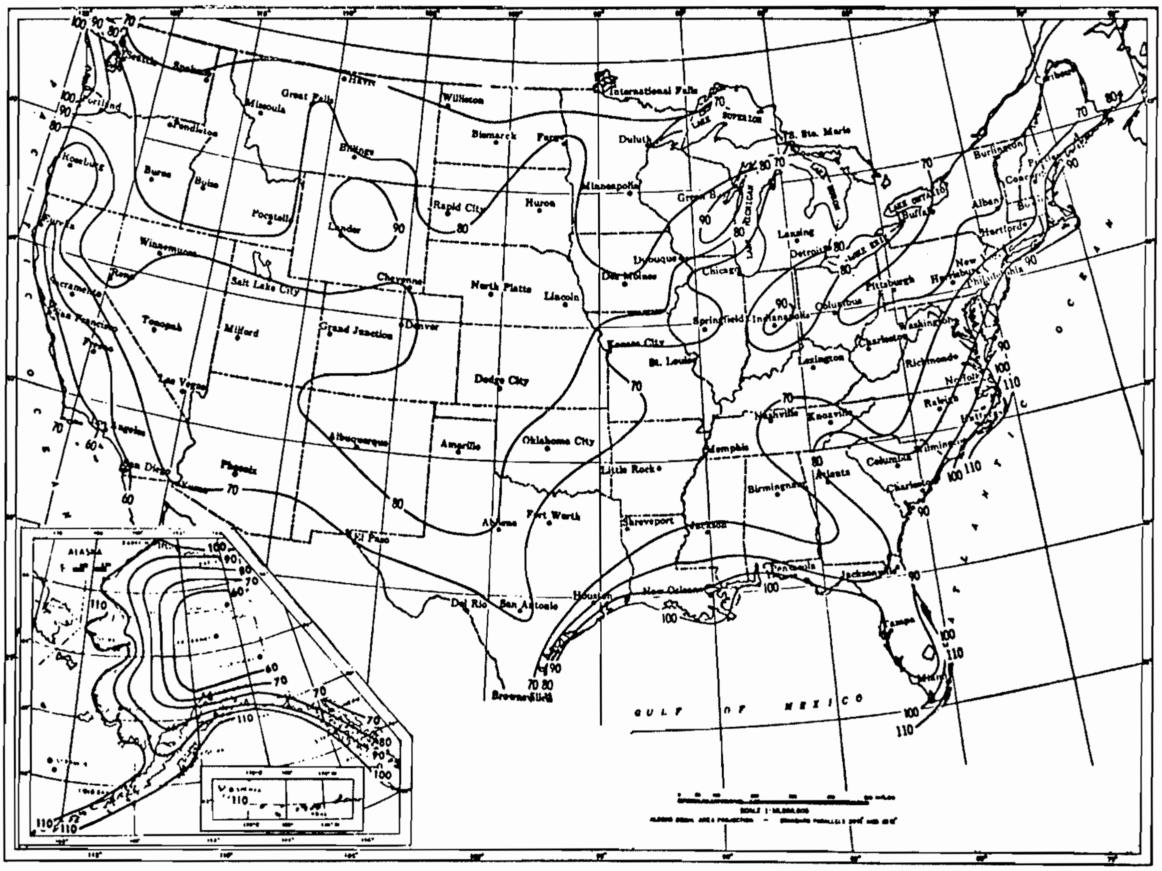
The projected area of nominal roof overhangs (as opposed to breezeways, patio roofs, etc.) may be neglected in determining wind loads. Construction subject to possible future enclosure such as breezeways and carports, however, should be designed for wind as if they were enclosed.

D. Overturning

Example design analyses in other parts of this report do not consider wind loads in checking overturning.

1. Where it is required that wind loads be used in checking for overturning and when wind loads govern, use horizontal wind load to the wall in lieu of seismic load in determining stability.

2. In those areas requiring load due to wind to be increased when considering overturning, multiply the horizontal wind load by 1.5 and use the resulting load to check for stability and design for overturning.



Basic Wind Speed in Miles Per Hour. Annual Extreme-Mile
30 Ft Above Ground, 50-Yr. Mean Recurrence Interval

FIGURE 3.9. Basic Wind Speed. Source ANSI A58.1 (Reproduced from MAP)

FIGURE 3.10. Minimum Design Wind Loads for Parts and Portions – Lateral Wind Pressure* on Windows, Etc. (Reproduced from MAP)

Exposure A, B, C

$$p = 0.8\,q_p - (-0.3)\,q_M = 0.8\,q_p + 0.3\,q_M;\ \text{psf}$$

Height (ft) H	Basic Wind Speed (mph)																	
	80			90			100			110			120			130		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
30 or less	15	15	24	15	18	31	15	23	38	15	27	46	17	33	55	19	29	64
50	15	17	27	15	22	34	15	26	43	18	32	52	21	38	62	25	45	73
100	15	22	32	16	27	40	20	34	50	25	41	60	30	49	72	35	57	84
150	16	25	35	20	31	44	25	38	54	30	46	67	36	55	79	42	65	92
200	19	27	37	23	34	48	29	43	59	34	52	71	41	62	85	48	73	99
250	21	30	39	26	38	50	32	47	61	39	56	75	46	67	89	54	78	105
300	23	32	41	29	40	52	35	49	64	43	60	77	51	71	92	60	84	108

*To be used if tributary areas are less than 200 sq ft (See note, page 1 of this Commentary)

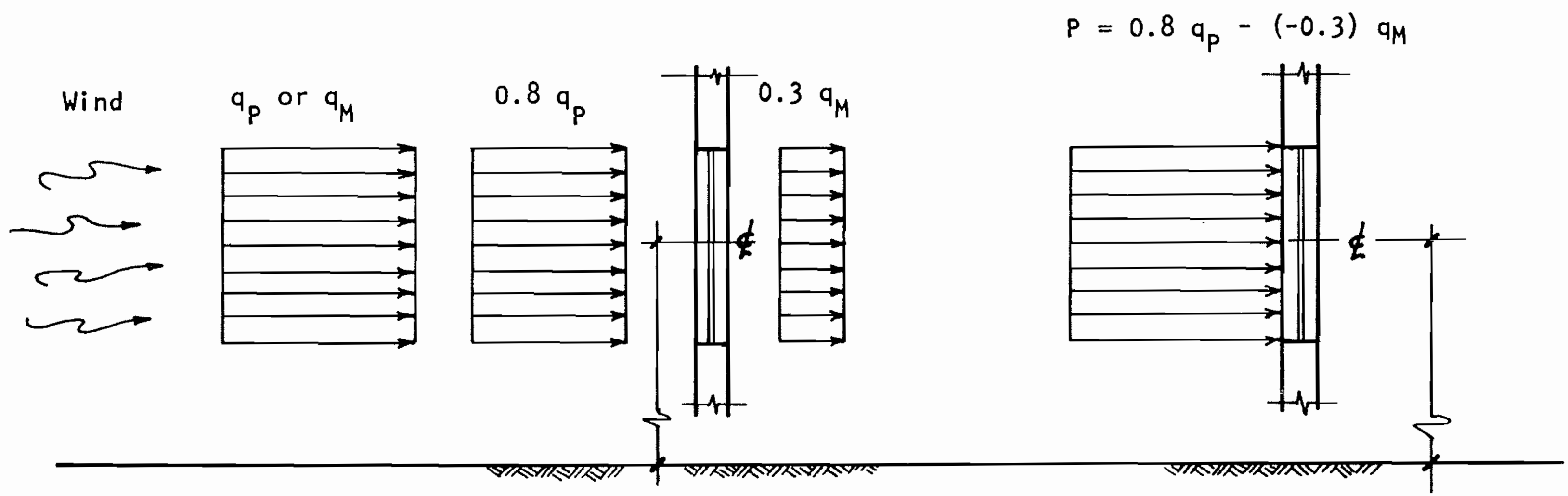
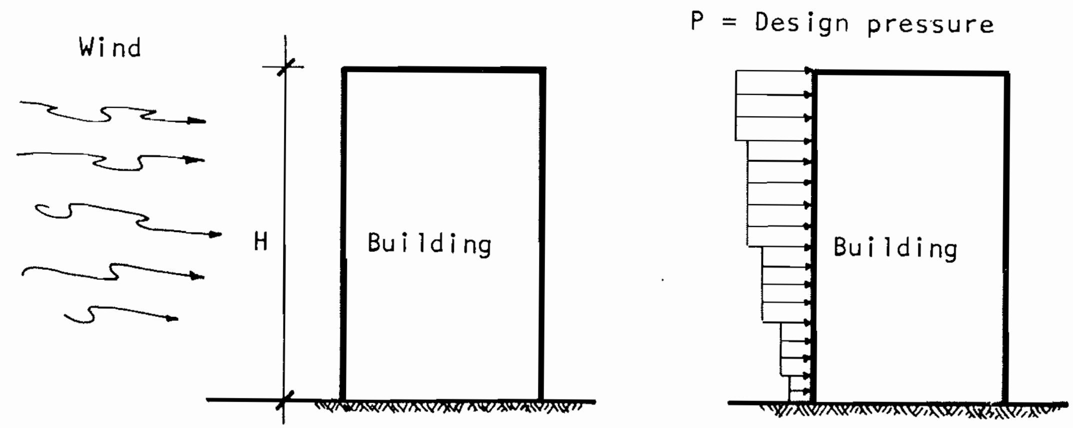


FIGURE 3.11. Minimum Design Wind Loads for Ordinary Buildings and Structures — Resultant External Pressures* — For Calculating Frame Action, Foundation Reactions, Etc. (Reproduced from MAP)

Exposure A, B, C
(p = 1.3 q_F); psf

Height (ft) H	Basic Wind Speed (mph) 80			90			100			110			120			130		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
Under 30	15	15	24	15	17	26	15	21	34	15	23	42	16	30	51	18	35	59
30	15	17	27	15	21	35	15	26	43	17	33	52	20	38	62	23	44	73
50	15	20	31	15	25	40	17	31	49	21	38	60	25	44	70	30	52	83
100	16	25	36	20	31	46	23	39	57	29	47	69	35	56	82	40	66	96
150	18	29	40	23	36	51	29	44	62	35	53	75	42	64	90	49	75	105
200	21	31	43	27	40	53	34	49	66	40	60	81	48	70	96	56	83	112
250	23	34	44	30	43	56	36	53	69	44	64	85	53	77	100	62	90	117
300	26	36	47	33	46	59	40	56	73	49	68	88	59	81	104	69	95	122

*To be used only if tributary areas are larger than 1000 sq ft (See Note, page 1 of this Commentary)



SECTION 6: WOOD FRAME SHEAR WALL DESIGN

Although other formats presenting the same information are acceptable, use of Calc Forms 8 and 9 is recommended for the design of all shear walls. In any case, shear wall design shall include determination of:

- wall shear
- sufficiency of shear-resisting materials provided, including fastener spacing
- sill bolt sizing and spacing
- wall stability against overturning

Calc Form 8 is for use in designing shear walls for seismic forces only. Calc Form 9 is for use in designing shear walls for both seismic and wind forces. It is the same as Calc Form 8 with additional columns provided for the determination of total wind load to each wall. (Method for determining wind load is set forth in Section 5.2 and is not discussed further in this section.) Instructions pertaining to Calc Form 8 also pertain to the appropriate columns of Calc Form 9.

6.1 Calc Form 8 – Shear-per-foot

Provide the required information in columns of Calc Form 8 as follows:

A. Line.

Indicate line of shear resistance by letter, number or verbal description. Where lines are considered to be acting together indicate them together.

B. Load.

Enter the total load for the line described from Calc Form 7.

C. Wall Length.

Enter the total effective length of all shear walls on the line of resistance or the lines acting together.

D. Shear-per-foot.

Shear-per-foot, as described in this report shall be used to design sill bolts (except when modified by Section 6.3B) and shear-resisting materials for walls of the same height (see Section 6.3B for walls of varying height). Shear-per-foot may be determined by dividing load by wall length from the two previous columns. Alternatively, approximate shear-per-foot may be obtained from Figures 3.12 through 3.15* as follows:

*Tables and Figures are presented at the end of Section 6.

6.1D 1. Enter figure with wall length.

2. Move upward on figure along wall length line to a point opposite the total load, matching the load in the second column of Calc Form 8 with the left side of the figure.

3. Read approximate shear-per-foot from diagonal line directly above the point determined.

When shear material being used is indicated on Figure 3.12, this graph should be used first to determine the adequacy of the material. When a more accurate determination of shear-per-foot is desired (for sill bolt design or use of shear materials or fastenings not indicated on Figure 3.12) the subsequent figures may be consulted in the same manner to obtain the most nearly accurate shear-per-foot.

6.2 Sill Bolt Design and Installation

A. Shear-per-foot

1. When the sill plate beneath a shear wall is the same length as the effective length of the shear wall, the shear-per-foot determined for the wall is the shear-per-foot for which the sill bolts shall be designed.

2. When the sill plate is longer than the shear wall (as when it extends under a window opening, etc.) and the height of the wall above the extended sill plate is a minimum of 2'-0" total sill plate length may be used in determining the shear-per-foot for sill bolt design.

   a. Shear-per-foot for the extended length of sill plate shall be determined in the same manner as described in Section 6.1D.

   b. When a "continuous" sill plate extends beneath two or more shear walls, the total load to those walls shall be divided by the total length of sill plate to determine the shear-per-foot to the sill plate.

   c. Sill plates may be considered "continuous" when the height of the wall above the sill plate is 2'-0" or more throughout its length and the provision of a properly fastened shear material is continuous. The plate itself need not be a single piece, but when abutted shall be bolted as indicated in Section 6.2C.

B. Spacing

Sill bolt spacing shall be determined from Figure 3.16. Typical size and spacing as well as special sizes and/or spacing for individual walls shall be specified on the plan for sill plates 6'-0" or more in length. When sill plates are shorter than 6'-0" the number and size of sill bolts for each such wall shall be specified. The table-matter in the upper right hand corner of Figure 3.16 may be used to make this determination. Sill bolt location shall be as specified in the following sections.

1. Determine sill bolt spacing for houses with slab on grade as follows:

CALC FORM 8

Job ______________________ Model ______________________

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material	Overturning

CALC FORM 9

Job ______________________ Model ______________________

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material	Overturning

6.2B1 a. Enter Figure 3.16 with shear-per-foot in the sill.

b. Read upward to curve representing size bolt desired.

c. Read horizontally to determine spacing required for sill bolts.

2. Spacing When Wood First-Floor Framing is Used

When wood first-floor framing is used, sill bolt spacing shall be determined in the following manner:

a. Determine total horizontal load to the mudsill from shear walls directly above the mudsill and from shear walls not connected to other continuous footings which fall within the first-floor tributary area of the mudsill under consideration.

b. Determine shear-per-foot by dividing total horizontal load by the length of the mudsill.

c. Add 50 pounds per foot for the shear created by the first-floor framing.

d. Determine size and spacing as indicated above.

C. Anchor Bolts

Foundation plates or sills shall be bolted to exterior foundations or foundation walls with anchor bolts (as detailed in Part IV) embedded at least 7 inches into concrete or reinforced masonry or at least 15 inches into unreinforced grouted masonry. The bolts shall be installed in accordance with the following requirements:

1. Bolts shall be not less than 1/2" in diameter.

2. Holes in sill plates shall be no more than 1/16" larger than the diameter of the bolts. Standard washers shall be used and nuts shall be tightened securely.

3. Centers of all anchor bolts shall be within 1/2" of the center line of the foundation plate or sill for 2 x 4 walls and within 1" of the center line for 2 x 6 or larger. Bolts placed closer than specified to the edge of the plate or sill may be left in place but a concrete stud anchor of the same size shall be installed within 12 inches of the improperly installed bolt.

4. Anchor bolts shall be spaced no more than 6'-0" apart (closer spacing shall be indicated on the plans) and shall be no closer to one another than 12" o.c.

5. There shall be a minimum of two bolts per piece.

6. A bolt shall be located a minimum of 6" and a maximum of 12" from each end of each piece. See also item 7 below.

7. When spacing requirements as set forth above are otherwise met bun bolts may

6.2C be used in lieu of anchor bolts to satisfy the requirement of a bolt at each end of plates or sills where they abut. The gun bolt shall not be used to fulfill spacing or minimum-bolts-per-piece requirements. Gun bolts used at abutting sill plates shall be placed a minimum of 6" from the nearest anchor bolt.

8. In the case of slab on grade construction, anchor bolts shall extend a minimum of 4 inches into the foundation or footing wall in addition to meeting all other pertinent requirements of this section.

9. At exterior corners of the house additional 12-inch anchor bolts shall be placed within 2-3/4 inches of the end studs as shown on Detail 24/4. These bolts shall be in addition to the other sill bolts and shall not be considered in meeting any of the requirements listed above.

10. Anchor bolts installed for interior non-bearing shear walls may be installed as shown on Detail 19/4 in lieu of the provision of a continuous footing. This detail shall not be used, however, when strap- or angle-type hold-downs are required at the end of the shear wall.

D. Gun Bolts (Power-driven Studs).

Foundation plates or sills may be bolted to interior foundations, foundation walls or slabs on grade with gun bolts in accordance with the following requirements:

1. Gun bolts shall be 7/32" round by 3-5/16" long with 3/8" minimum head diameter. A 16 gage by 2" round washer or 3/32" by 3/4" round washer shall be used with each gun bolt.

2. There shall be a minimum of three bolts per piece with one bolt located at each end of each piece as specified under Anchor Bolts.

3. Bolts shall be placed within 1/2" of the center line of the plate or sill.

4. Gun bolts shall be spaced not more than 3'-0" apart nor less than 6" o.c. Spacing of gun bolts shall be determined using Figure 3.16 as described for Anchor Bolts. When spacings closer than 3'-0" o.c. are required, the required spacing for each such wall shall be specified on the plans.

5. Gun bolts shall not be used at exterior walls except as indicated in Section 6.2C7 and shall be installed at interior walls only as allowed by local building departments and other controlling agencies.

6. Gun bolts may be installed at interior non-bearing walls as shown on Detail 20/4.

6.3 Shear Material Design and Installation

A. Shear-per-foot

Enter the value of the shear material used at a given wall or group of walls on Calc Form 8 or 9 above the line designating those walls. Compare the shear-per-foot (or the adjusted shear-per-foot as instructed in 6.3B below) with the allow-

6.3A able value for the material used to determine the sufficiency of the shear wall. Enter a notation for each wall under the column headed Shear Material indicating the acceptability of the designated shear material or the appropriate material to be used on a given wall.

B. Adjusted Shear-per-foot

Shear walls of varying heights along the same line of resistance or along lines acting together shall have the shear in the walls adjusted as follows:

1. Obtain a ratio by dividing the height of each shear wall by its length.

2. Add the ratios for all walls to obtain a total of the ratios.

3. Divide each wall's ratio by the total of the ratios to obtain a secondary ratio. The total of the secondary ratios should equal 1.000, with error only in the last number behind the decimal point.

4. Multiply total horizontal load by each of the secondary ratios. The resultants represent the horizontal load at each of the shear walls.

5. Divide horizontal load at each wall by the wall's length to obtain shear-per-foot.

6. Use the maximum shear-per-foot obtained to design shear-resisting materials for <u>all</u> the walls.

C. Installation of Shear-Resisting Materials

1. General Requirements

   a. Methods of attachment for shear-resisting finish materials set forth in the Minimum Property Standards are applicable for Zone 1. All attachments and shear-resisting values for Zones 2 and 3 shall be as shown in Tables 3.9 and 3.10.

   b. Shear values tabulated for the various materials listed shall not be additive with the values for other materials applied to the same wall. Shear values may be doubled when identical material is applied as specified in this section to both sides of a wall.

   c. Finish materials for all exterior walls and designated interior shear walls shall be installed as specified herein. Other interior walls shall have their finish materials applied in accordance with these requirements but stud size or spacing may be at variance with that specified.

2. Materials Other Than Plywood

   Shear-resisting materials other than plywood shall be installed and have allowable shear values as shown in Table 3.9.

   a. Diagonal sheathing shall be made up of 1″ nominal sheathing boards at an angle of approximately 45° to the studs and shall be installed with not more than a 1/2" space between boards. Sheathing boards shall be nailed directly to each intermediate stud or other framing member with not fewer than two 8d nails for 1" by 6" nominal boards and not fewer than three 8d nails for board 8" or wider; in addition, 6" boards shall be nailed with three 8d nails and 8" boards with four 8d nails at the shear wall boundaries. End joints of adjacent boards shall be separated by at least one stud space and there shall be at least two boards between joints on the same stud.

b. Gypsum sheathing board shall be fastened as shown in Table 3.9. 2'-0" x 8'-0" sheathing board may be unblocked; 4'-0" wide pieces must be blocked. 4'-0" wide pieces may be applied parallel or perpendicular to studs. 2'-0" wide pieces must be applied perpendicular to studs.

c. Gypsum wallboard may be applied parallel or perpendicular to studs. Where required, blocking shall have the same cross-sectional dimensions as the studs, and shall be provided at all joints that are perpendicular to the studs.

d. Fiberboard sheathing shall be applied vertically. Blocking of not less than 2" nominal thickness shall be provided at horizontal joints when such joints occur at locations other than at the top and bottom plates. Where siding is to be fastened to fiberboard sheathing, nail-base sheathing shall be used. Fiberboard or nail-base sheathing shall conform to the latest adopted or revised ASTM specifications, C208 and D2277, respectively.

e. Stucco—Plastering with Portland Cement plaster shall not be less than three coats when applied over metal lath or approved wire fabric lath. All lath and lath attachments shall be of corrosion-resistant materials. Backing is not required under metal lath or paperbacked wire fabric lath. The first coat shall be applied with sufficient material and such pressure to fill all openings in the lath. The surface shall be scored horizontally sufficiently rough to provide adequate bond to receive the second coat. The second coat shall be brought out to 3/4" in total thickness, rodded and floated sufficiently rough to provide adequate bond for finish coat. The finish coat shall be applied with sufficient material and pressure to bond to and to cover the brown (second) coat and shall be of sufficient thickness to conceal the brown coat. Maximum volume of sand per volume of cement shall not exceed 4:1 for the first coat, 5:1 for the second coat, and 3:1 for the finish coat. The first and second coats shall each be moist-cured for a minimum of 48 hours. Finish coat shall not be applied until seven days after the second coat.

f. Gypsum lath and plaster. Gypsum lath shall be applied with the long dimension perpendicular to studs or other supports and with end joints staggered in successive courses. Where lath edges are not in moderate contact and have joint gaps exceeding 3/8", the joint gaps shall be covered with stripping or cornerite. Stripping or cornerite may be omitted when the entire surface is reinforced with not less than 1" No. 20 U.S. Gage woven wire. Plastering shall be not less than two coats and shall be gypsum plaster. Base coats shall be applied with sufficient material and pressure to provide a complete key or bond. The first coat shall be brought out to grounds and straightened to a true surface, leaving the surface rough to receive the finish coat. First coat shall be applied with sufficient material and pressure to form a complete bond. Thickness of finish coat shall be not less than 1/16".

6.3C2 g. Hardboard shall be 4 x 8 sheets applied vertically. Blocking of not less than 2" nominal thickness shall be provided at horizontal joints when such joints occur at locations other than at the top and bottom plates. Hardboard materials used shall be approved by HUD Acceptance Procedures and shall meet the requirements of Appendix D of the HUD Minimum Property Standards as well as Voluntary Product Standard PS600-73.

3. Plywood

Plywood shall be installed and have allowable shear values as shown in Table 3.10. All plywood siding and plywood applied to the exterior side of shear walls covered with porous finishes shall be Exterior type. All plywood used for shear walls shall meet the requirements of "Product Standard No. 1 (PS 1) for Softwood Plywood – Construction and Industrial." Panel thickness and application details shall be as given below and in Part IV of this report.

a. Grading and Sizing

Structural I is limited to Group 1 species. Each panel shall bear the stamp of a qualified inspection and testing agency as further defined by US Product Standard PS1-74. The designations Structural I and PS 1-74 will appear on the stamped marking of all plywood sheets of this grade.

Structural II. Although Structural II is a designated plywood grade, the designation as used herein shall be interpreted as any plywood stamped by a qualified inspection and testing agency bearing a stamp indicating its compliance with PS 1-74 but not stamped as Structural I or as Siding. All such plywood shall have allowable shear values of 90% of those indicated for Structural I plywood, as indicated by the center grouping in Table 3.10. When Structural I plywood is designated on the plans or on the details in Part IV, "Structural II" plywood as herein defined may be substituted for Structural I plywood but the required thickness must be increased by 1/8" and nail size must conform to that indicated by Table 3.10 for the thickness of plywood actually used.

Siding. Plywood siding shall be nailed as indicated in the bottom portion of Table 3.10 and shall be stamped as siding and as indicated above. Siding shall have a minimum thickness of 3/8" unless placed over 1" wood sheathing or 1/2" plywood sheathing.

b. Application to Stud Walls

All plywood sheathing shall be installed with vertical joints over studs and horizontal joints nailed to 2" nominal blocking or continuous plates. Nail size shall be as indicated in Table 3.10 for the thickness of plywood used. Interiors of sheets shall be nailed to intermediate framing members with nails spaced at 12" o.c. When nail spacing is 2-1/2" o.c. or less, 3" nominal studs and blocking shall be provided at all plywood edges. Plywood may be installed either vertically or horizontally as indicated by the diagrams at the bottom of Table 3.10.

Methods of fastening other than those indicated in Table 3.10 may be used in accordance with HUD-FHA Bulletin No. UM-25d. The method of fasten-

6.3C3 ing used shall have the same or greater allowable shear than is required by the nailing indicated on the plans.

4. Edges and Ends

All edges and ends of sheet-type finish material shall occur on the framing members, except those edges and ends that are perpendicular to the framing members when blocking is not required. Blocking shall always be provided for edges and ends perpendicular to framing members for fiberboard sheathing, hardboard, plywood and 4'-0" wide pieces of gypsum sheathing board. Nailing shall be spaced not less than 3/8" from edges and ends of sheet-type materials and, in addition, shall be nailed as specified in Table 3.9 or 3.10 to all intermediate studs and blocking.

5. Sizes

Sheet-type materials shall be installed such that no piece is less than 2'-0" x 4'-0" except that when the distance between, above or below openings is less than 2'-0" each piece shall be installed to its largest possible dimensions.

6.4 Overturning

Length of walls as described in this subsection shall be used only for determining overturning stability and shall not be applied to other determinations in these design recommendations.

Overturning stability shall be determined in accordance with the following requirements:

A. Length of Walls

Definition of "Solid Walls" – when sufficient wall occurs above or below an opening adjacent to a shear panel and when that opening is not so long as to make the wall above or below it too limber for use, the shear wall, the width of wall containing the opening, and the wall at the opposite side of the opening may be considered as acting as a unit to resist overturning, may be considered as if it were solid. In other words, when the opening is not so large as to preclude the shear wall and the adjacent wall from acting as a unit to resist overturning, the entire assembly is defined as a "solid" wall and may be treated as if the opening did not exist.

1. Conditions

A wall may be considered "solid" when the following conditions exist:

a. The height of the opening within the wall shall not exceed 55% of the height of the wall (4'-3" for 8'-0" nominal height walls).

b. A minimum shear material height of 2'-6" must be provided above or below the opening. The shear-resisting material may not be interrupted within the 2'-6" minimum height by holes, pipes, etc., larger than 6 inches high and 14 inches wide, located with the edge of the hole 6 inches minimum from the top or bottom of the wall section.

6.4A1

c. The width of the opening may not exceed 2.5 times the height of the highest section of wall above or below. In no case shall such openings be greater than 8'-0" in width.

d. A full-height wall panel at the far side of the opening need not qualify (in width) as a shear wall in order to be considered a part of the "solid" wall.

2. Two or More Openings

In order for a wall to be considered "solid" past two or more openings, the conditions at each opening must be met as described above. In addition, the width of wall between openings must be equal to the highest section of wall above or below the openings. The wall between the openings must be continuous and uninterrupted from the sill plate to the top plates.

a. In the case of openings with a height of wall above or below the opening of more than 4'-0", the width of the wall between openings need not exceed 4'-0".

b. When the highest section of wall at each of two adjacent openings differs, the width of wall required between them shall be determined from the lesser of the two heights.

c. When the width of wall between openings is less than that specified above, the "solid" wall may be considered as extending to the far side of the wall between openings but may not be extended farther.

3. Posts

When an opening is broken by a post supporting the header over the opening, the post may be considered as wall provided that all other requirements contained herein are met. Shear-resisting material below the opening shall be nailed to the post with the edge-nailing required for the material installed. This provision assumes that the 2'-6" minimum height required occurs below the opening. When the opening dimensions will qualify (ignoring the post), the post need not be considered. When the opening width would be too great, the "solid" wall shall be considered to stop at the far side of the post.

When two or more posts occur, the length of the wall may be extended to the farthest post allowed by other requirements contained herein, as if the intermediate posts did not exist.

Example: Window 10'-0" wide with posts supporting the header at the third points (3'-4") adjacent to 6'-0" long shear panel; wall below window 2'-8" high; wall above window 1'-4" high. Applying Condition c., 2.5 x 2'-8" = 6'-8" and therefore wall can be extended to second post for a total length of shear wall of 6'-8" + 6'-0" = 12'-8". Note that if a shear wall occurs on the far side of the opening a similar extension could be made for that shear wall. In this case the center 3'-4" section would be used for each shear wall in determining its length for overturning resistance, but the two walls cannot be considered together, thereby further extending the length to include both walls.

6.4A 4. Two-Story Construction

When the shear material is continuous and in the same plane past the second-floor framing of two-story construction, the total height of the first-story wall (for the purposes of these requirements) may be considered as the height to the sills of the windows of the second floor or, where no such openings exist, to the top plates at the roof line, and percentage of opening height may be determined using this increased height.

a. Plywood, hardboard and fiberboard may be considered continuous when their abutting edges are nailed to the same horizontal framing member.

b. When shear-resisting materials are not continuous past the second-floor framing, each floor must be considered independently.

B. Vertical Loads (No Calc Forms are provided for determination of vertical load.)

Vertical dead loads contributing to the resistance of shear walls to overturning shall be determined as follows:

1. Weight of veneer shall not be considered as contributing to the resistance of overturning; therefore veneer weight shall not be included in vertical load determination.

2. Use the same wall weight as determined on Calc Form 1 or 3 or 10 psf minimum.

3. Use roof and ceiling loads as determined on Calc Form 1 or 2 (actual loads).

4. When framing is parallel to the shear wall under consideration, use second floor load as determined on Calc Form 2. When framing is perpendicular to the shear wall under consideration, add wall weight in pounds per square foot (1 sq ft per sq ft of second floor) to second floor dead load.

5. Vertical load per lineal foot of wall shall be made up of the following components as applicable:

   a. Wall weight per square foot times height of wall.

   b. One-half of each span of rafters or roof trusses spanning to the wall (plus any overhang) times the roof dead load.

   c. One-half of each span of ceiling joists supported by the wall times ceiling dead load.

   d. One-half of each span of floor joists supported by the wall times second-floor load including partition load.

6. Where two or more dead loads occur along a single length of wall the lesser of the loads shall be used in determining overturing resistance for that wall.

7. When an interior shear wall on the first floor of a two-story house has a parallel wall directly above it or within 3'-0" of it at the second floor,

6.4B7 the truss above the second floor non-bearing wall may be considered as contributing load to the interior wall as if the truss were supported by this wall. When this option is used, loads to interior and exterior walls shall be figured as if the interior wall were a bearing wall thereby reducing the load to adjacent exterior walls.

8. Exterior walls at gable roofs may be considered the same height as the walls perpendicular to them at each end, provided the top plates of the walls being considered are continuous (except for any break at the ridge). When considered at this lower height for purposes of determining resistance to overturning, that same height shall be used in calculating vertical load.

C. Horizontal Load Capacity

Allowable horizontal load for each length of wall along a line of shear resistance shall be determined by consulting the "No Hold-Downs Required" graphs presented in this section. Where vertical loads vary from the vertical loads presented, interpolations shall be made between graphs. A "solid" wall shall be considered to be a single length of wall regardless of the number of shear panels contained within it. Where vertical load exceeds 600 lb/ft, allowable horizontal load may be determined by adding together the allowable horizontal loads for various increments of the total vertical load-per-foot.

D. Total Overturning Resistance

Overturning capacity along a line of resistance shall be determined as follows:

1. Total the allowable horizontal loads for each length of wall along the line of resistance as determined from the "No Hold-Downs Required" graphs.
2. Add 100 pounds horizontal load for each length of wall occurring along the line.
3. The resulting summation is the total horizontal force the walls are capable of resisting without installation of hold-down anchors.
   a. If the summation is no more than 5% less than the actual horizontal load, the wall may be considered stable and no further design is required.
   b. When horizontal force is greater than the resistance offered with no hold-down anchors, the formula:

$$U = \frac{H}{L} \times (P_{act} - P_{all})$$

shall be applied,

where H = height of wall
L = total length of all overturning segments
P_{act} = actual horizontal force along the line of resistance
P_{all} = allowable force as determined from the summation above
U = uplift load generated

1. If U equals 400 pounds or less, 1 framing anchor hold-down per Detail 26/4 may be used. If U equals more than 400 pounds but less than 800 pounds, a 2 framing anchor hold-down may be used.

6.4D3b

2. If U equals more than 800 pounds but less than 1700 pounds, strap hold-downs may be used.

3. If U is greater than 1700 pounds, angle hold-downs must be installed. Consult the graphs for hold-down Nos. 1, 2, 3 and 4 consecutively until the sum of the horizontal forces for each length of wall is greater than the actual horizontal force. Install the hold-down indicated at each end of each wall length.

The formula stated above should not be used for determining load to angle hold-downs.

4. Grade Beams

Grade beams shall be utilized with strap and angle hold-downs, as indicated in this subsection.

a. When strap hold-downs are installed place one # 4 bar each in the top and the bottom of the footing extending 6'-0" beyond each end of the shear wall.

b. When angle hold-downs are used, determine size of grade beam and number and size of reinforcing bars from the appropriate hold-down graph. Read directly upward on the wall length line until the line intersects the horizontal line indicating size and number of bars and size of grade beam. Then consult Table 4.2 in Detail 31/4 to determine "a" distance for size and number of reinforcing bars indicated. Details 29/4, 30/4 and 31/4 indicate "a" distance and placement of bars with cross-sections shown on 32/4 and 33/4.

c. When grade beams provided are deeper than those detailed, the area of steel may be reduced by applying the formula:

$$A = \frac{d_s}{d_a} \times A_s$$

where d_s = depth from top of grade beam detailed to center line of bottom reinforcing, as shown on Detail 32/4 or 33/4, whichever is applicable

d_a = actual corresponding depth for footing to be used

A_s = area of reinforcing as shown on details

A = area of reinforcing steel required to be installed top and bottom in the grade beam

d. When hold-down anchors are installed into 8'-0" high concrete basement walls, no special horizontal reinforcing is required. For masonry walls two # 4s or one # 5 top and bottom may be used with any hold-down.

e. Installation of "straight grade beams as shown on Detail 29/4 are the most efficient. Where such installations are not possible or are not economical, the grade beam may turn the corner as shown on Detail 30/4 or may be extended to an intersecting perpendicular footing at which point the footing may be developed as a grade beam in order to meet the minimum "a" distance (Detail 31/4). When such intersection forms a "T" in plan view, the reinforcing should be extended in both directions with laps at the intersection similar to that shown on Detail 30/4.

6.5 Overturning at Two-Story High Shear Walls

The overturning requirements above presuppose the existence of finish material above the head of first floor openings extending to the sill of second-floor openings (approximately 4'-0" minimum height from head to sill). Where shear walls have adjacent full-height openings at the first and second floor such that the glass is interrupted only by the second-floor framing (or a relatively small height of finish material), the wall receives no support from intervening shear-resistant material at the second floor line. Such walls function in overturning as if their height (as shown on the hold-down graphs) were their full two-story height but with loads at the roof and second floor line.

When such walls are common in a residential design, it is recommended that structural engineering be provided. Where one or two such walls occur, overturning of the walls may be evaluated using the procedures outlined above through use of the formula:

$$P_{10} = \frac{P_r h_r + P_2 h_2}{10}$$

where P_{10} = the equivalent horizontal load which would be applied at the top of a ten-foot-high wall

P_r = the seismic or wind load at the roof level

h_r = the height from the base of the shear wall (at the first floor) to the roof plate line

P_2 = the seismic or wind load at the second floor line

h_2 = the height from the base of the shear wall to the second floor

After determining the equivalent horizontal force applied to a 10'-0" high wall, design the wall for overturning using standard design procedures set forth in Section 6.4, assuming the wall to be 10'-0" high and its actual width. No "Solid" wall increases in length for overturning resistance are possible with this type of shear wall.

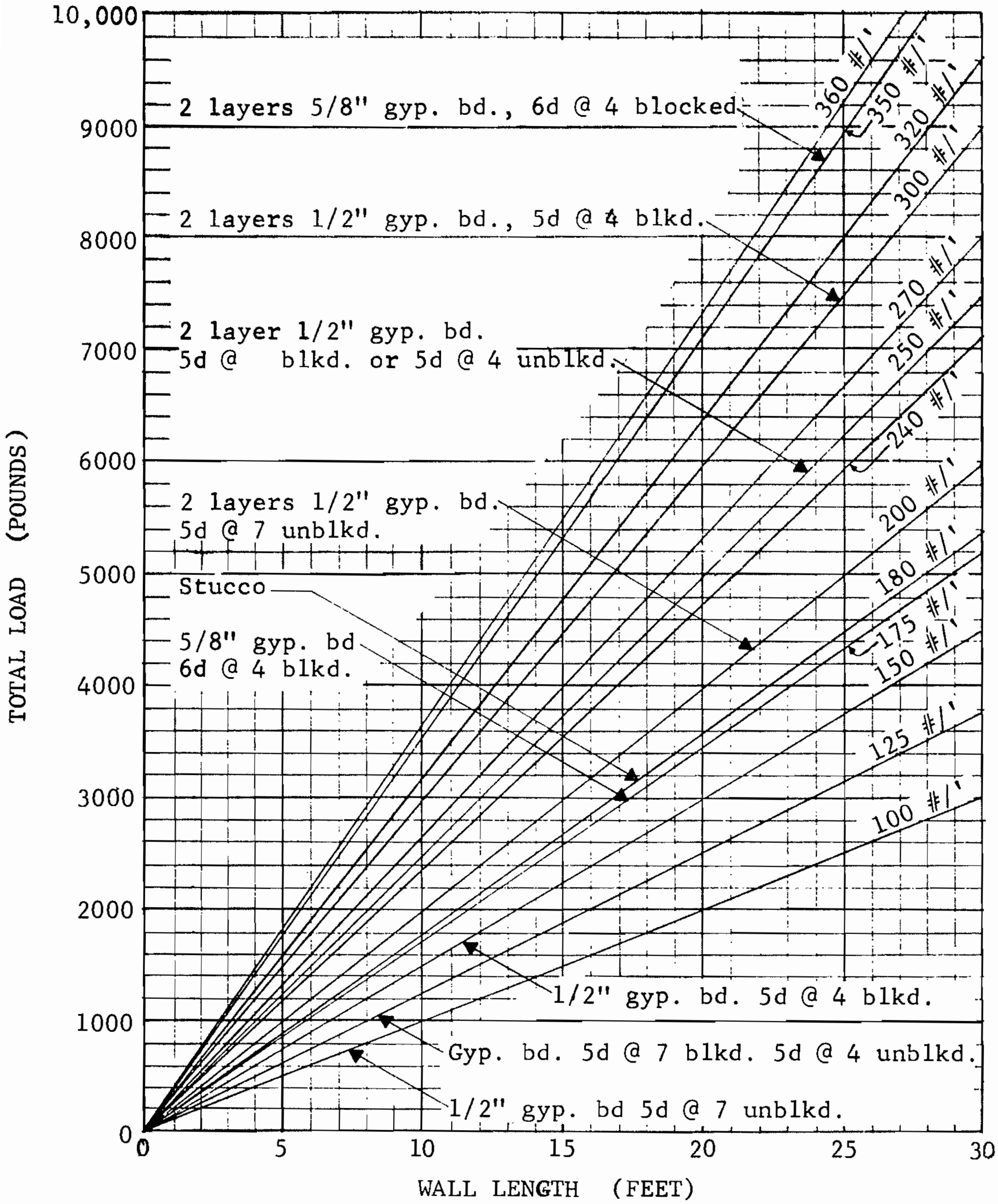


FIGURE 3.12 Total Horizontal Load Capacity for Walls With Various Shear-Resisting Materials

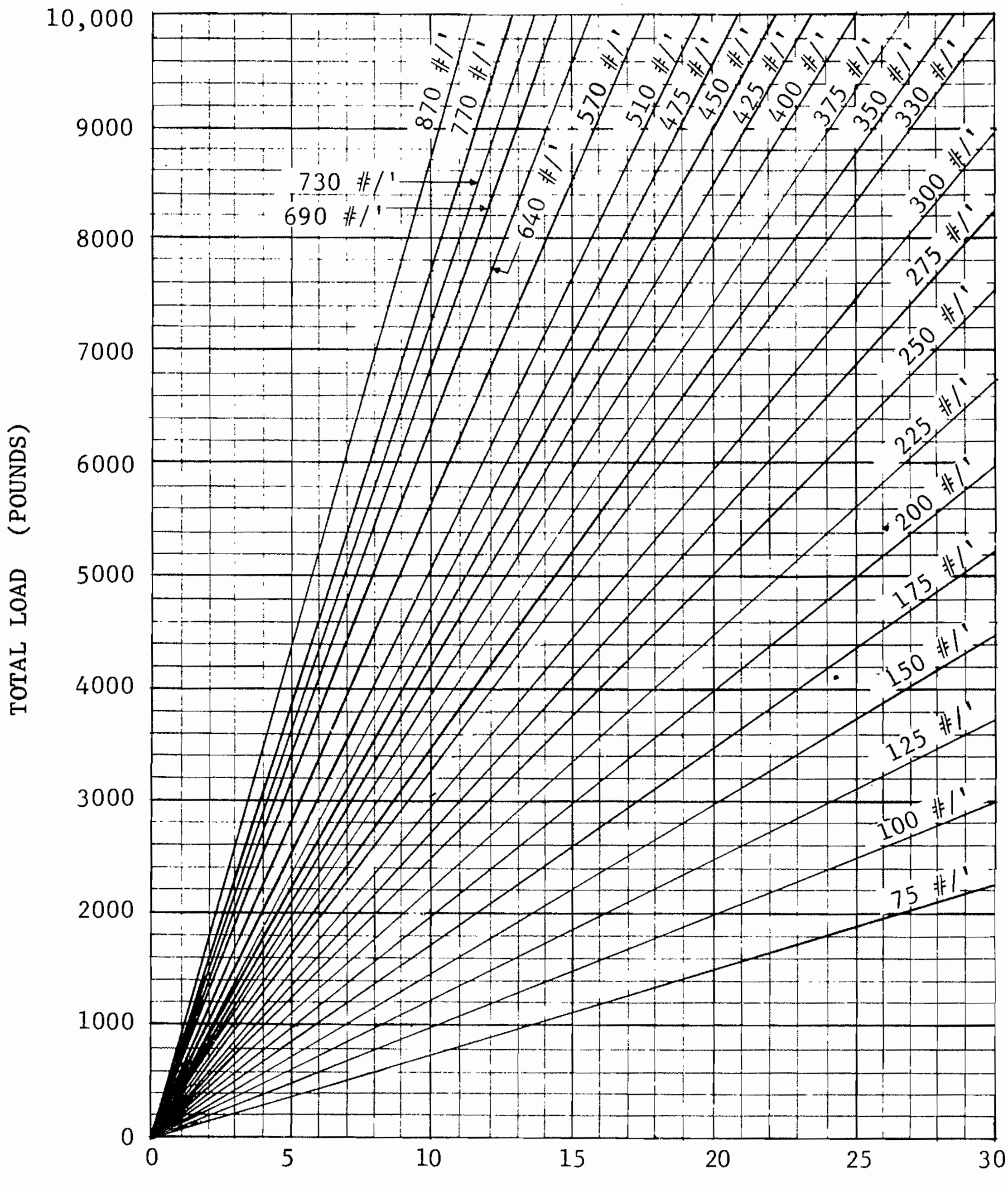


FIGURE 3.13 Total Horizontal Load Capacity for Walls With Various Shear-Resisting Materials

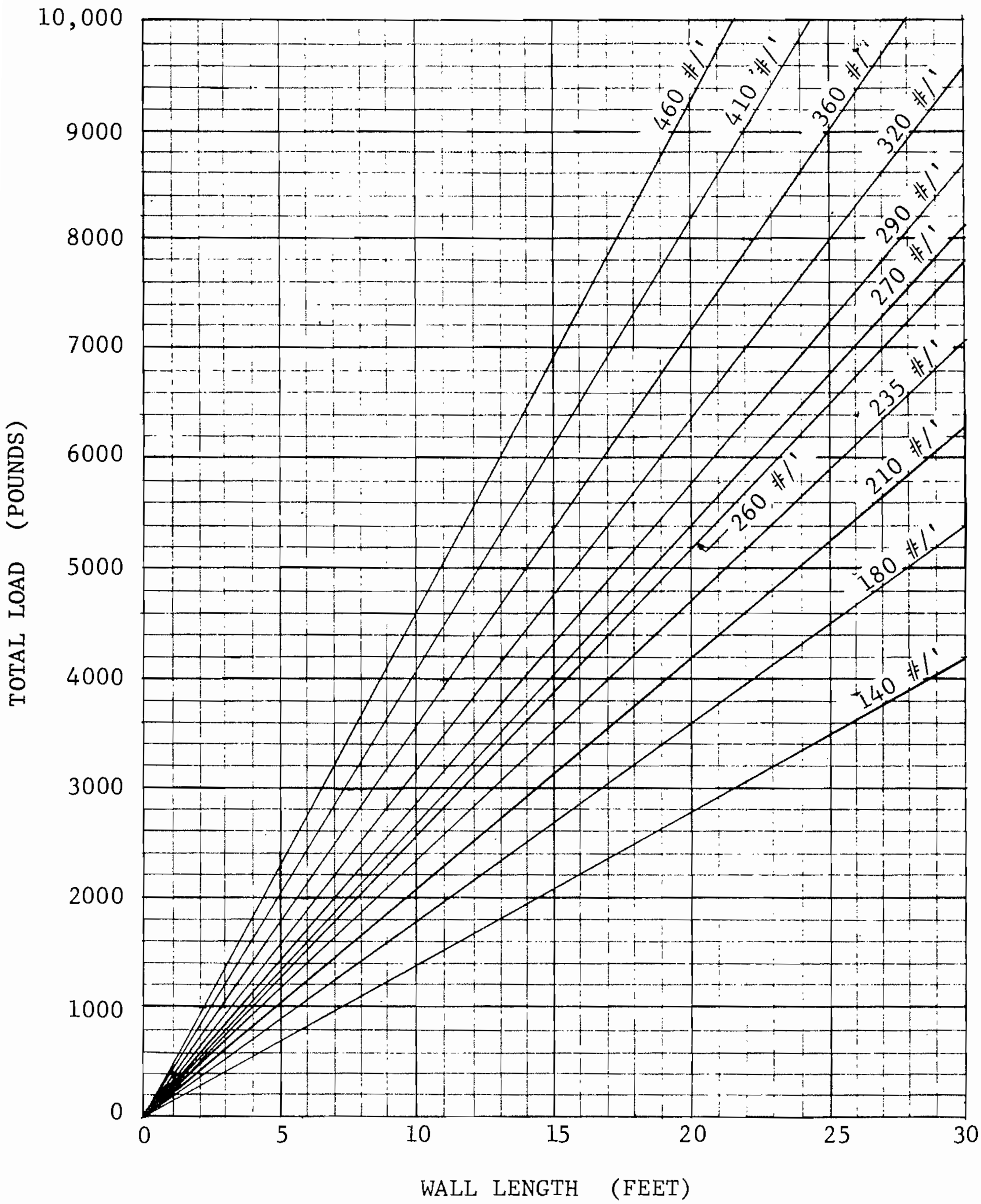


FIGURE 3.14 **Total Horizontal Load Capacity for Walls With Various Shear-Resisting Materials**

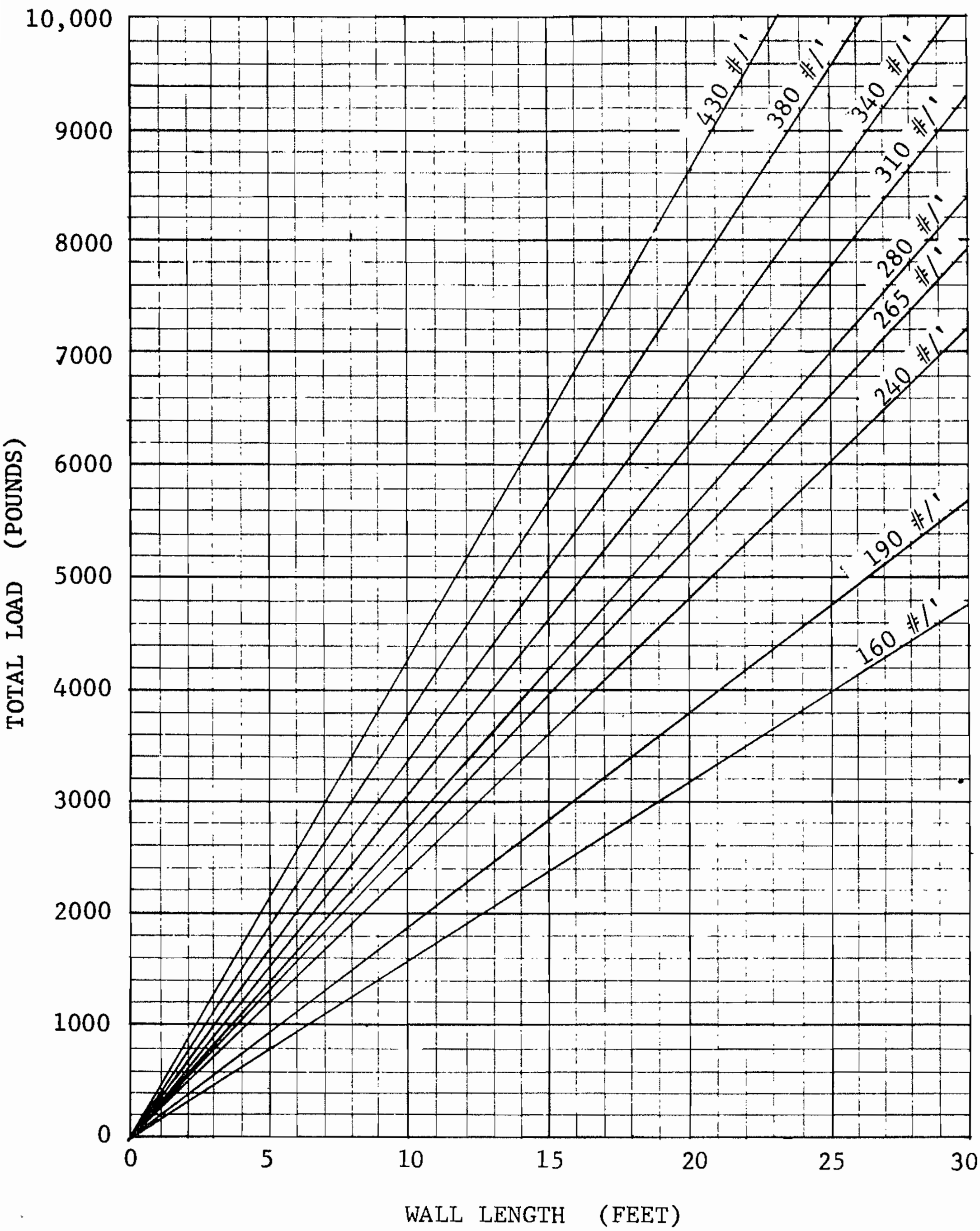
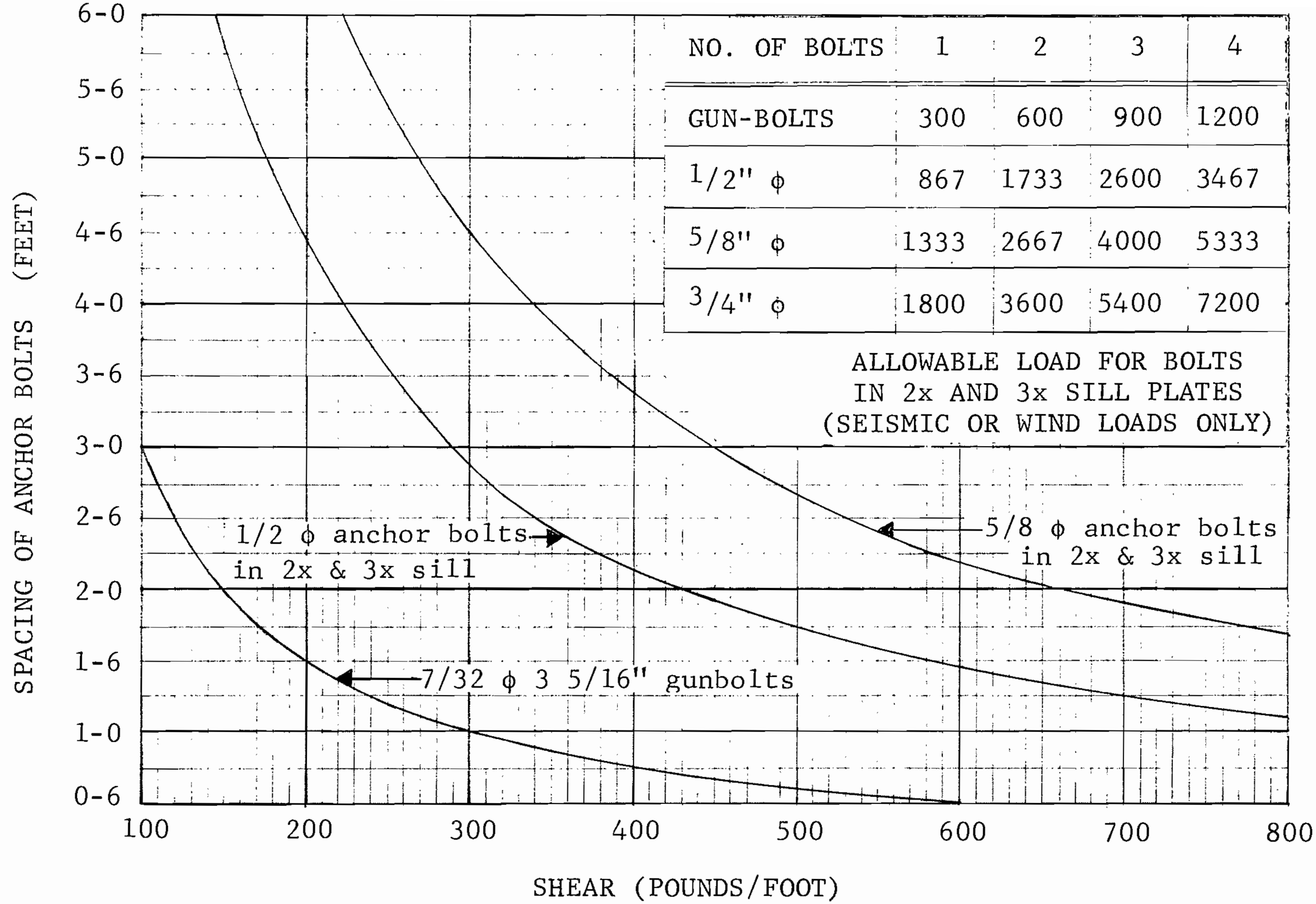


FIGURE 3.15 Total Horizontal Load Capacity for Walls With Various Shear-Resisting Materials



NO. OF BOLTS	1	2	3	4
GUN-BOLTS	300	600	900	1200
1/2" ϕ	867	1733	2600	3467
5/8" ϕ	1333	2667	4000	5333
3/4" ϕ	1800	3600	5400	7200

FIGURE 3.16 Sill Bolt Spacing

TABLE 3.9

ALLOWABLE SHEAR IN POUNDS PER FOOT FOR SINGLE-MATERIAL SHEAR WALL ASSEMBLIES WITH 2 x 4 OR LARGER STUDS AT 16" o.c.

MATERIAL	THICKNESS	WALL CONSTRUCTION	MAXIMUM NAIL SPACING(1) (INCHES)	SHEAR VALUE	MINIMUM NAIL SIZE
1" x 6" Diagonal Sheathing	3/4"	Unblocked	2 per board interior 3 per board boundary	300	8d common
1" x 8" Diagonal Sheathing	3/4"	Unblocked	3 per board interior 4 per board boundary	300	8d common
Gypsum Sheathing Board	1/2" x 2' x 8'	Unblocked	4	75	No. 11 gage 1-3/4" long, 7/16" head, diamond-point, galv.
	1/2" x 4'	Blocked	4	175	
Gypsum Wallboard	1/2"	Unblocked	7	100	5d cooler or 5d (.086" wire dia.) x 1-5/8" long, 9/32" concave head gypsum board nail or GWB-54 (.098 gage, 1-1/4" long, 1/4" head, annular ring)
			4	125	
		Blocked	7	125	
			4	150	
	5/8"	Blocked	4	175	6d (corresp. to above)
Fiberboard ASTM Designation C208 or D2277	7/16" x 4' x 8'	Applied vertically. Blocked	3" at all edges 6" at interior of sheets	125	No. 11 gage galv. roofing nail 1-1/2" long, 7/16" head
	25/32" x 4' x 8'			175	No. 11 gage galv. roofing nail 1-3/4" long, 7/16" head
	1/2" x 4' x 8' Nailbase			175	No. 11 gage galv. roofing nail 1-1/2" long, 7/16" head
Stucco Woven or Welded Wire Lath and Portland Cement Plaster	7/8"	Unblocked	6	180	No. 11 gage 1-1/2" long with 7/16" diameter head nail or No. 16 gage staples having 7/8" long legs
Gypsum lath plain or perforated	3/8" Lath and 1/2" Plaster	Unblocked	5	100	No. 13 gage 1-1/8" long, 19/64" head, plasterboard blued nail
Hardboard(2)	7/16"(3)	Applied vertically. Blocked	4" at all edges 8" at interior of sheets	230 shiplap vert. joints	6d box, galv.
				300 butted vert. joints	

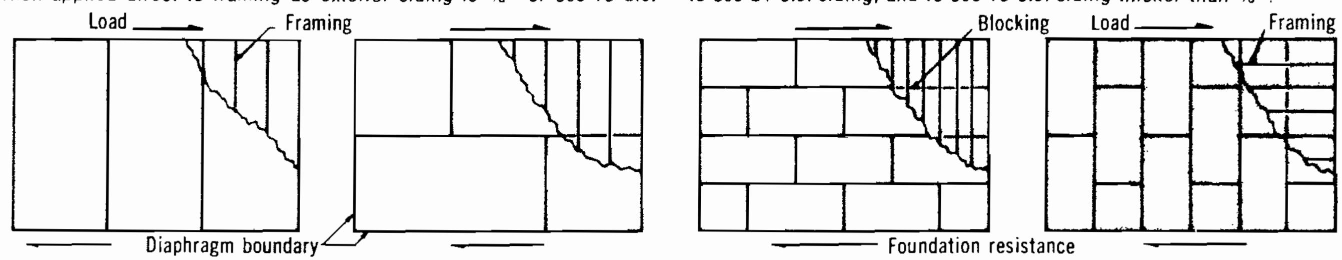
(1) Applies to nailing at all studs, top and bottom plates, and blocking.
(2) For all hardboards meeting requirements of Appendix D of HUD Minimum Property Standards and Voluntary Product Standard PS60-73.
(3) May be notched to 1/4" nominal thickness for architectural effects.

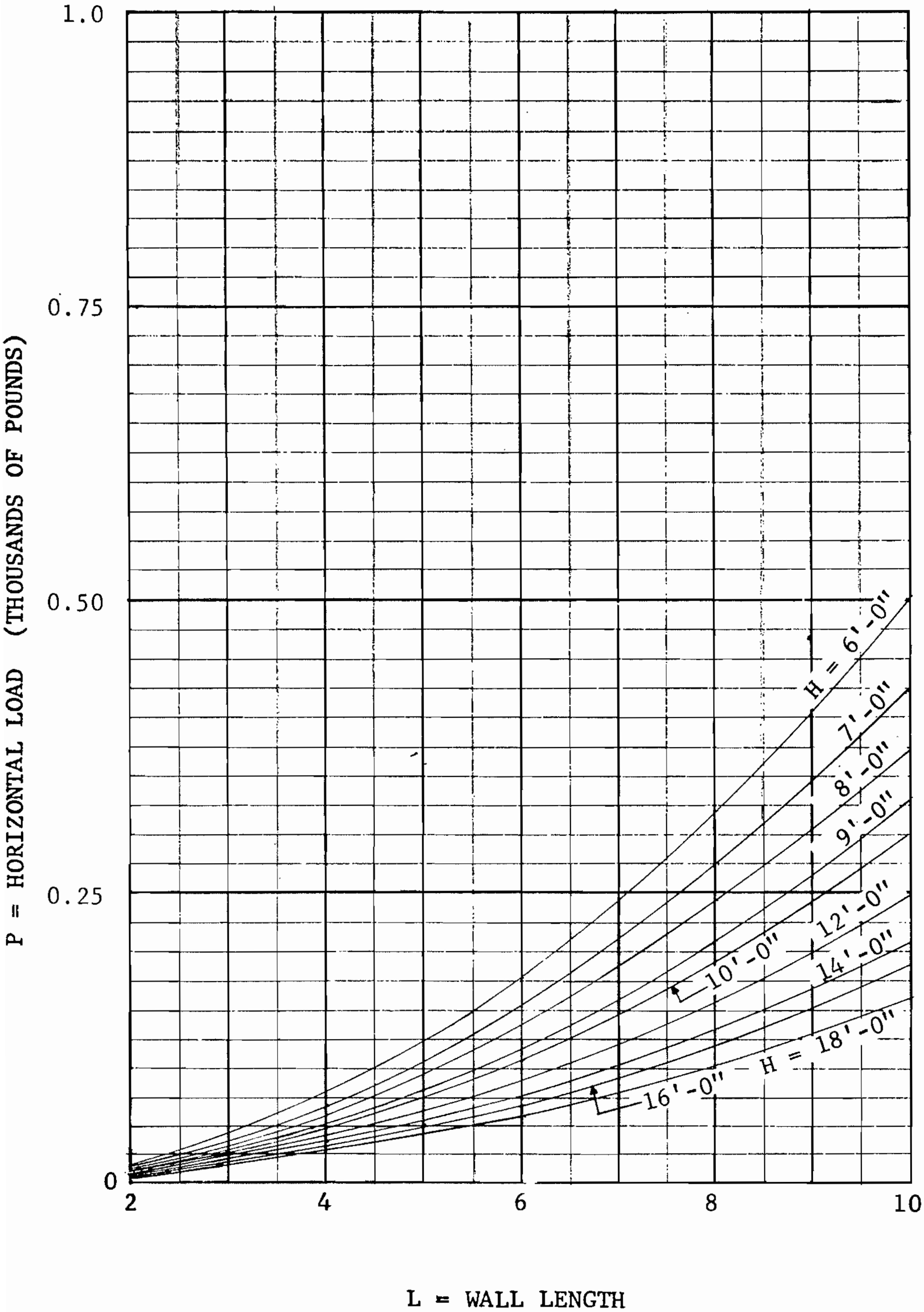
Recommended shear in pounds per foot for plywood shear walls (TABLE 3.10)

for wind or seismic loading [a]

Plywood Grade	Minimum Nominal Plywood Thickness (inches)	Minimum Nail Penetration in Framing (inches)	PLYWOOD APPLIED DIRECT TO FRAMING: Nail Size (Common or Galvanized Box)	Nail Spacing at Plywood Panel Edges (inches): 6	4	2½	2	PLYWOOD APPLIED OVER ½" GYPSUM SHEATHING: Nail Size (Common or Galvanized Box)	Nail Spacing at Plywood Panel Edges (inches): 6	4	2½	2
STRUCTURAL I C-D INT-DFPA or STRUCTURAL I EXT-DFPA	5/16 or ¼(b)	1¼	6d	200	300	450	510	8d	200	300	450	510
	⅜	1½	8d	280	430	640	730	10d	280	430	640(d)	730(d)
	½	1⅝	10d	340	510	770(d)	870(d)	—	—	—	—	—
C-C EXT-DFPA, STRUCTURAL II INT-DFPA, STANDARD C-D INT-DFPA, DFPA Panel Siding, and other DFPA grades(c)	5/16 or ¼(b)(e)	1¼	6d	180	270	400	450	8d	180	270	400	450
	⅜(f)	1½	8d	260	380	570	640	10d	260	380	570(d)	640(d)
	½	1⅝	10d	310	460	690(d)	770(d)	—	—	—	—	—
			Nail Size (Galvanized Casing)					Nail Size (Galvanized Casing)				
DFPA Plywood Panel Siding applied with casing nails(c)	5/16(b)(e)	1¼	6d	140	210	320	360	8d	140	210	320	360
	⅜(f)	1½	8d	160	240	360	410	10d	160	240	360	410

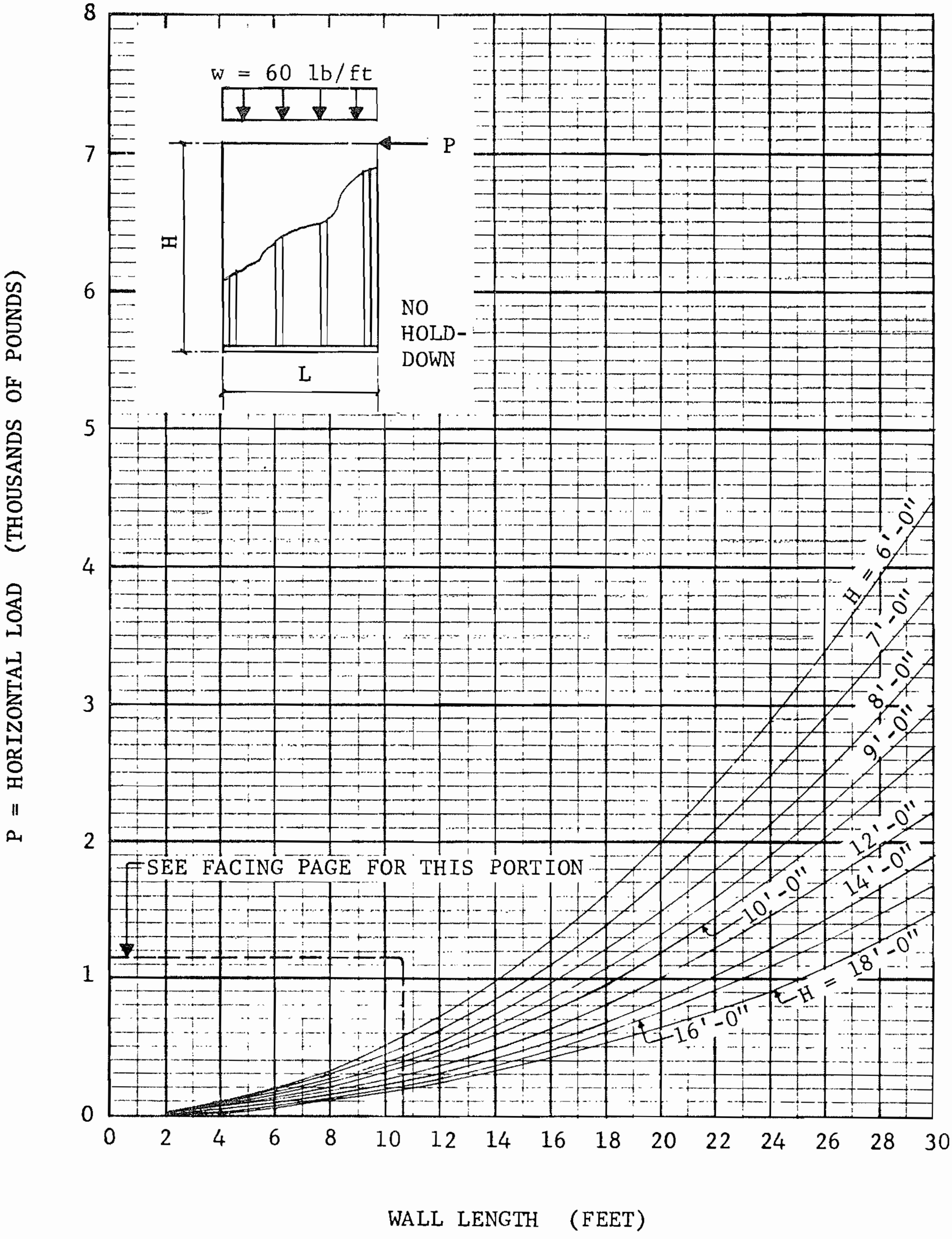
(a) All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 12 in. on center along intermediate framing members. (b) Minimum recommended when applied direct to framing as exterior siding is ⅜" or 303-16 o.c. (c) Except Group 5 species. (d) Reduce tabulated shears 10% when boundary members provide less than 3-inch-nominal nailing surface. (e) Applies also to 303-16 o.c. siding, ⅜" and thinner. (f) Applies also to 303-24 o.c. siding, and to 303-16 o.c. siding thicker than ⅜".





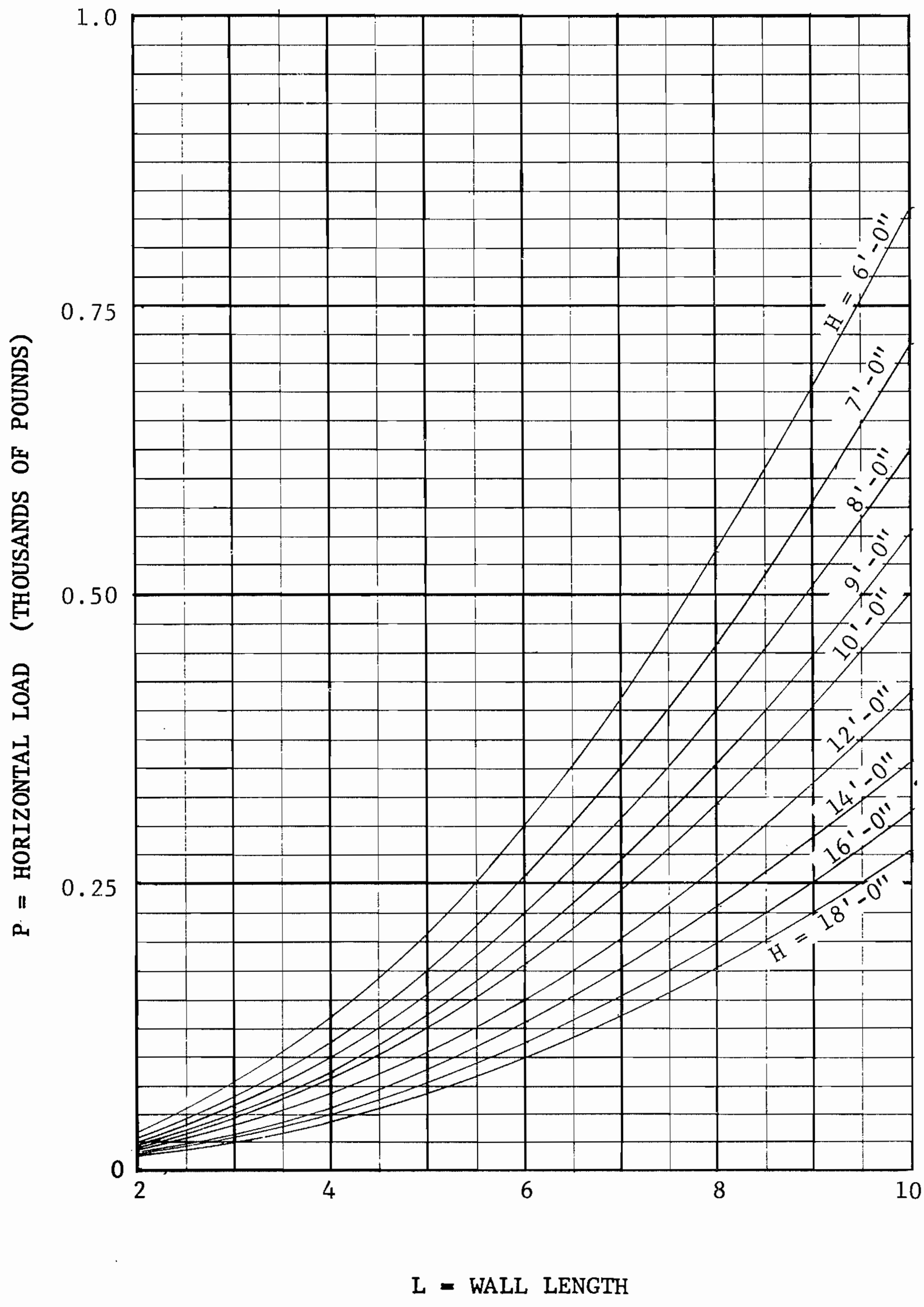
HOLD-DOWN GRAPH A

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 60 lb/ft



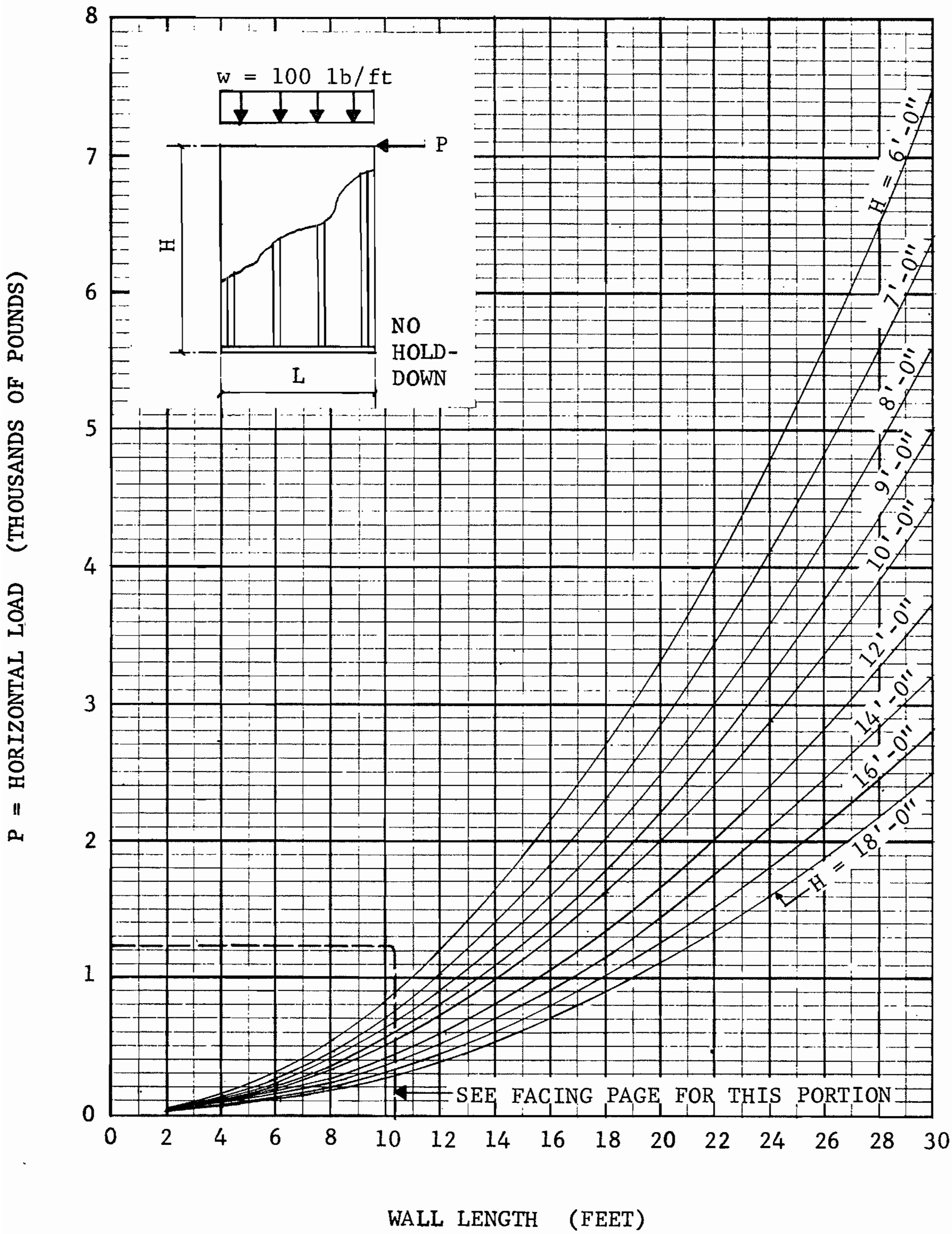
HOLD-DOWN GRAPH A

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 60 lb/ft



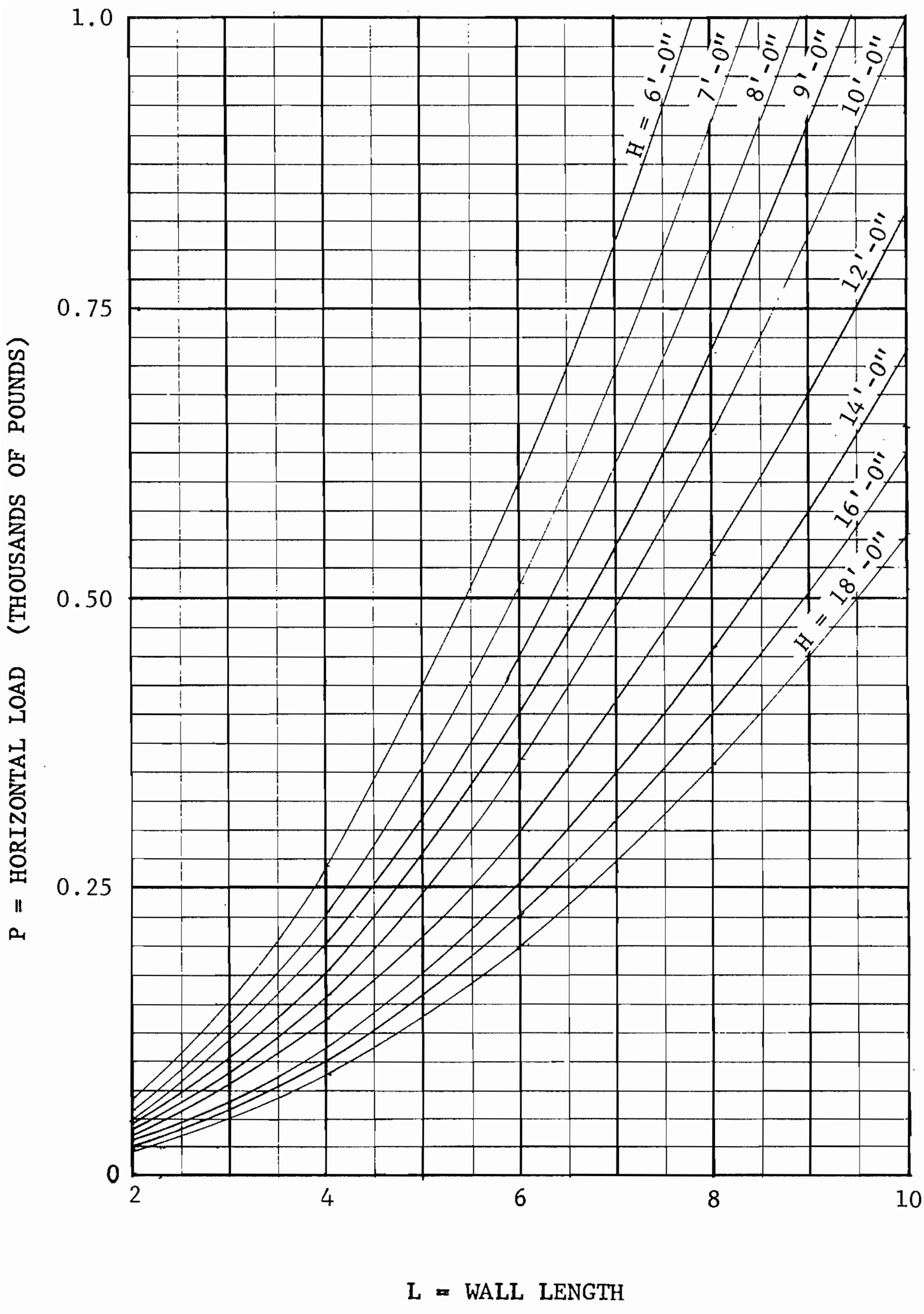
HOLD-DOWN GRAPH B

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 100 lb/ft



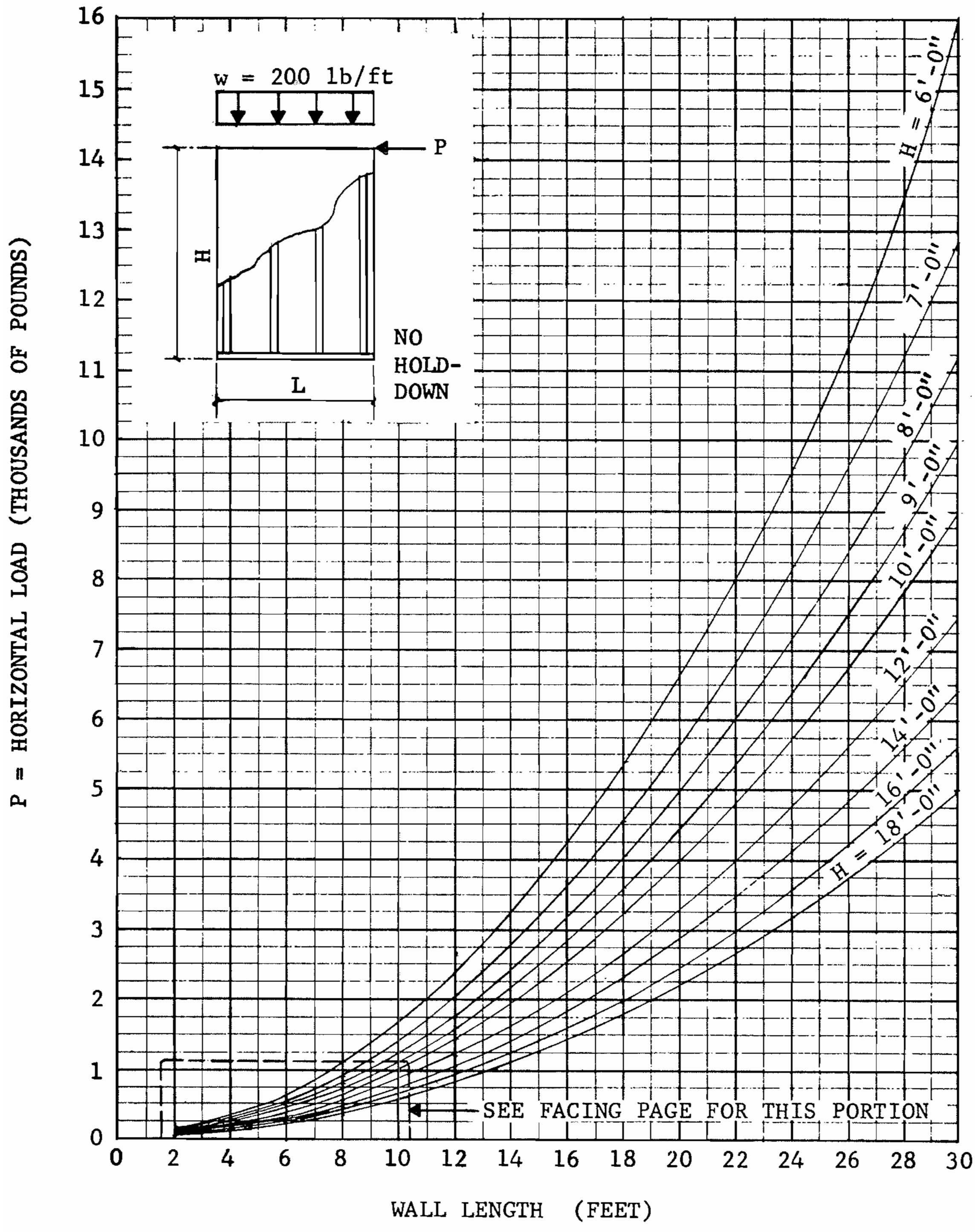
HOLD-DOWN GRAPH B

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 100 lb/ft



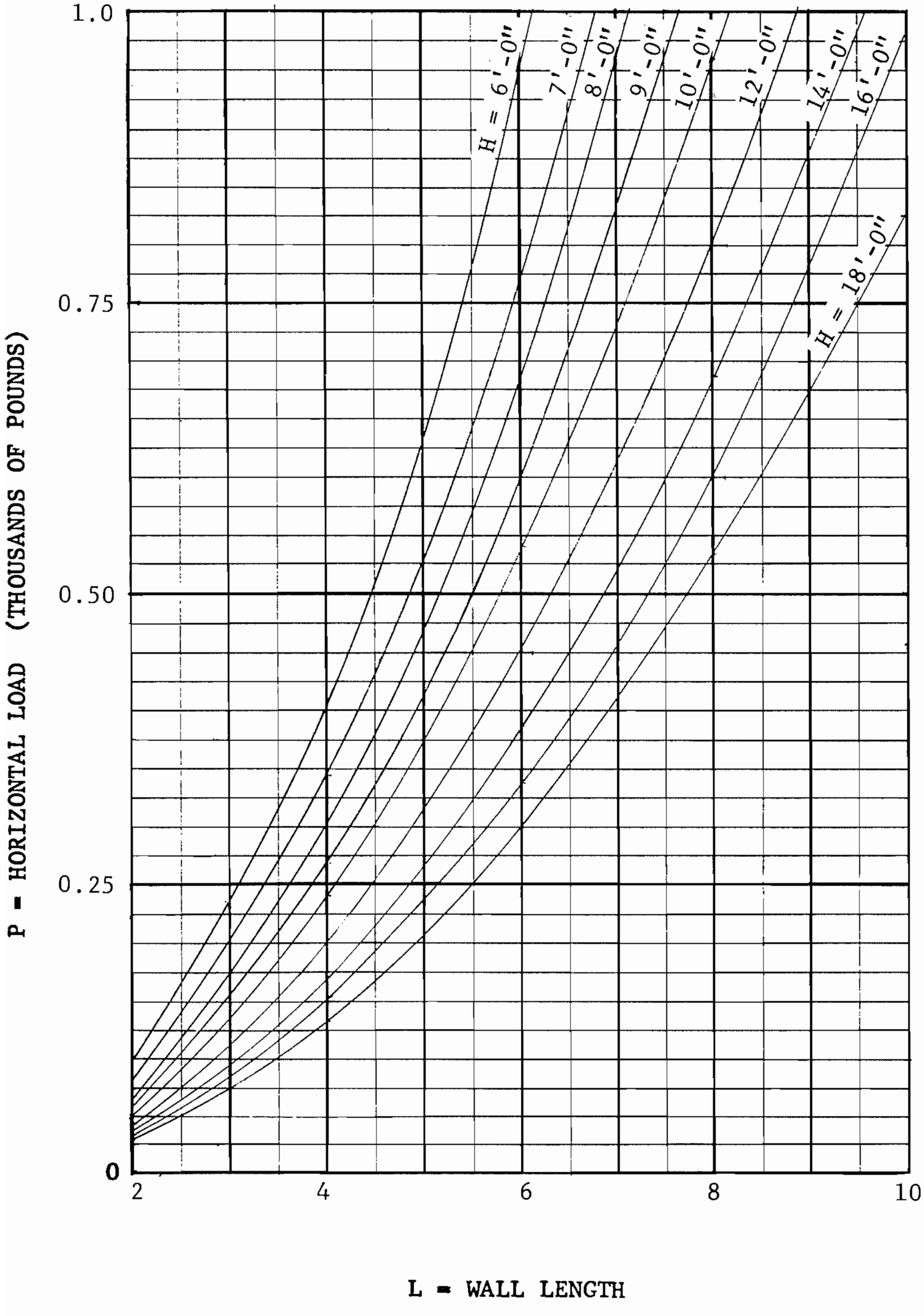
HOLD-DOWN GRAPH C

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 200 lb/ft



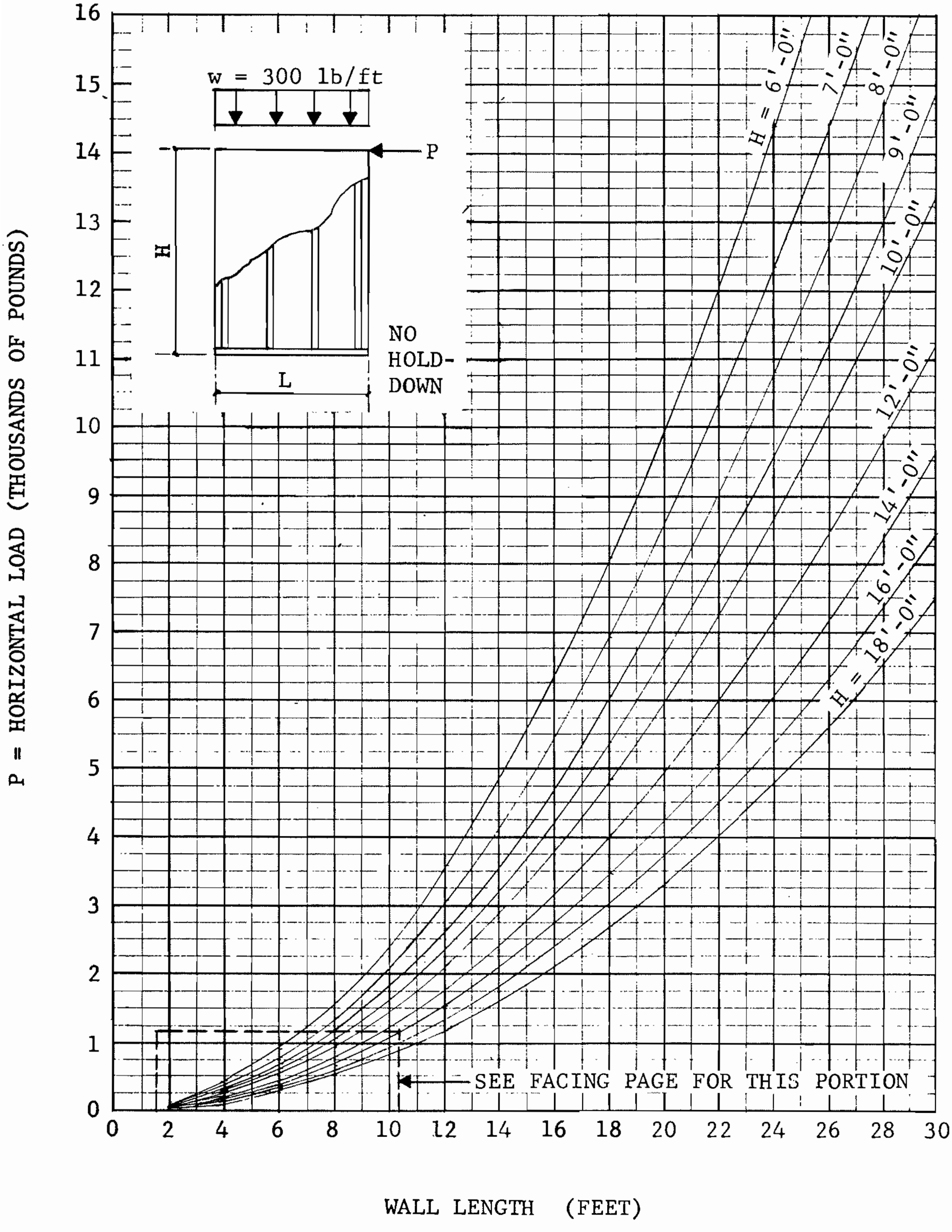
HOLD-DOWN GRAPH A

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 200 lb/ft



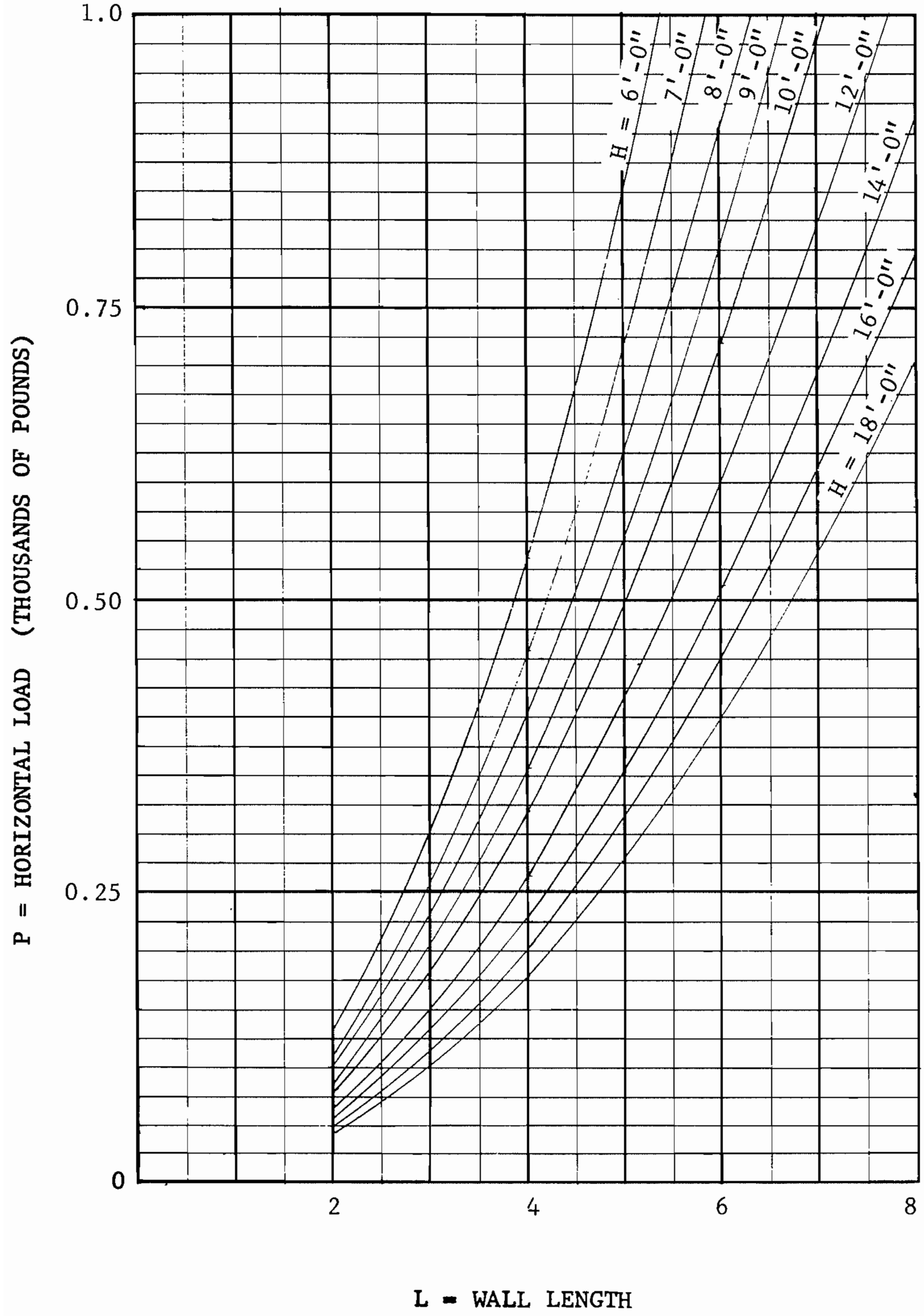
HOLD-DOWN GRAPH D

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 300lb/ft



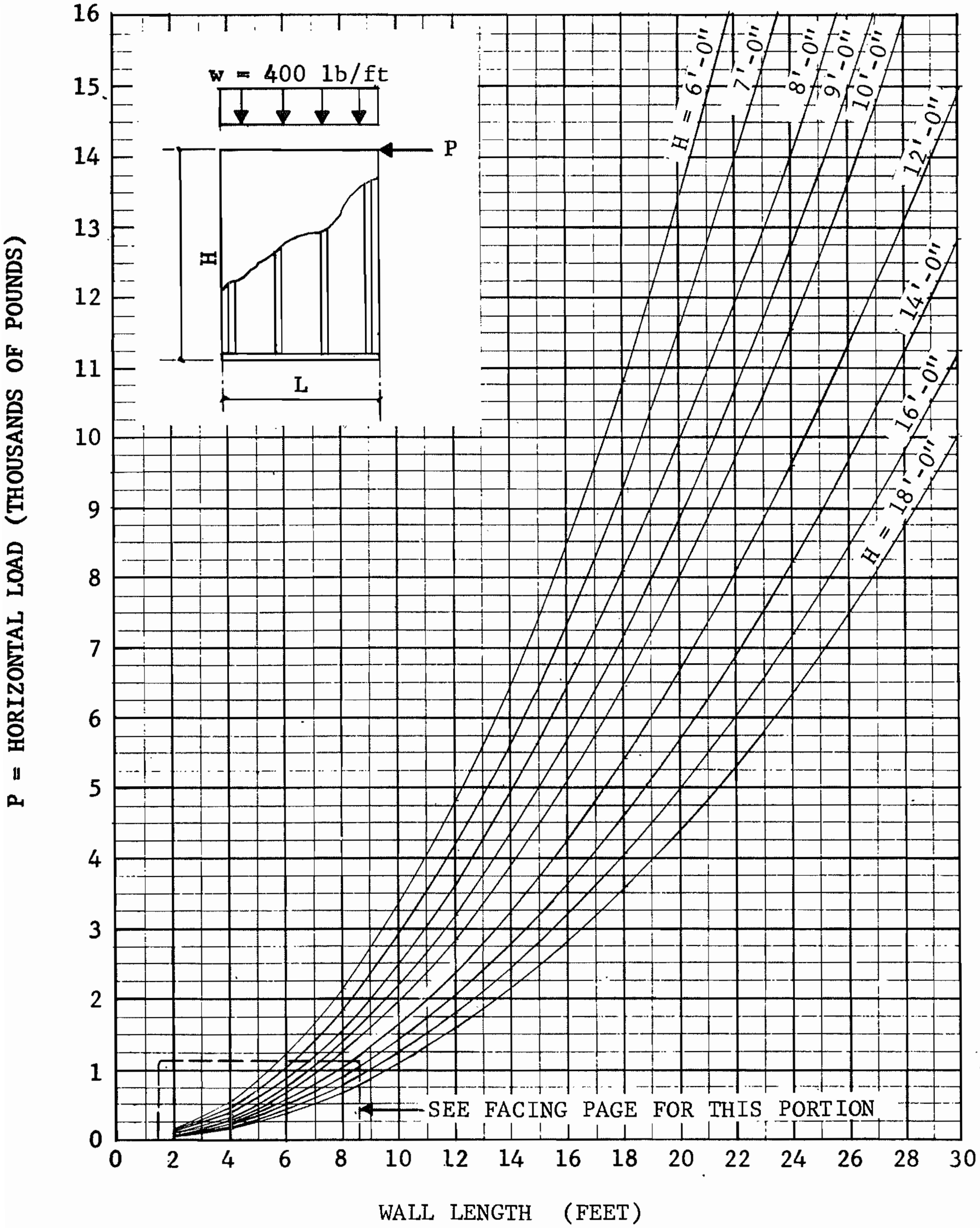
HOLD-DOWN GRAPH D

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 300 lb/ft



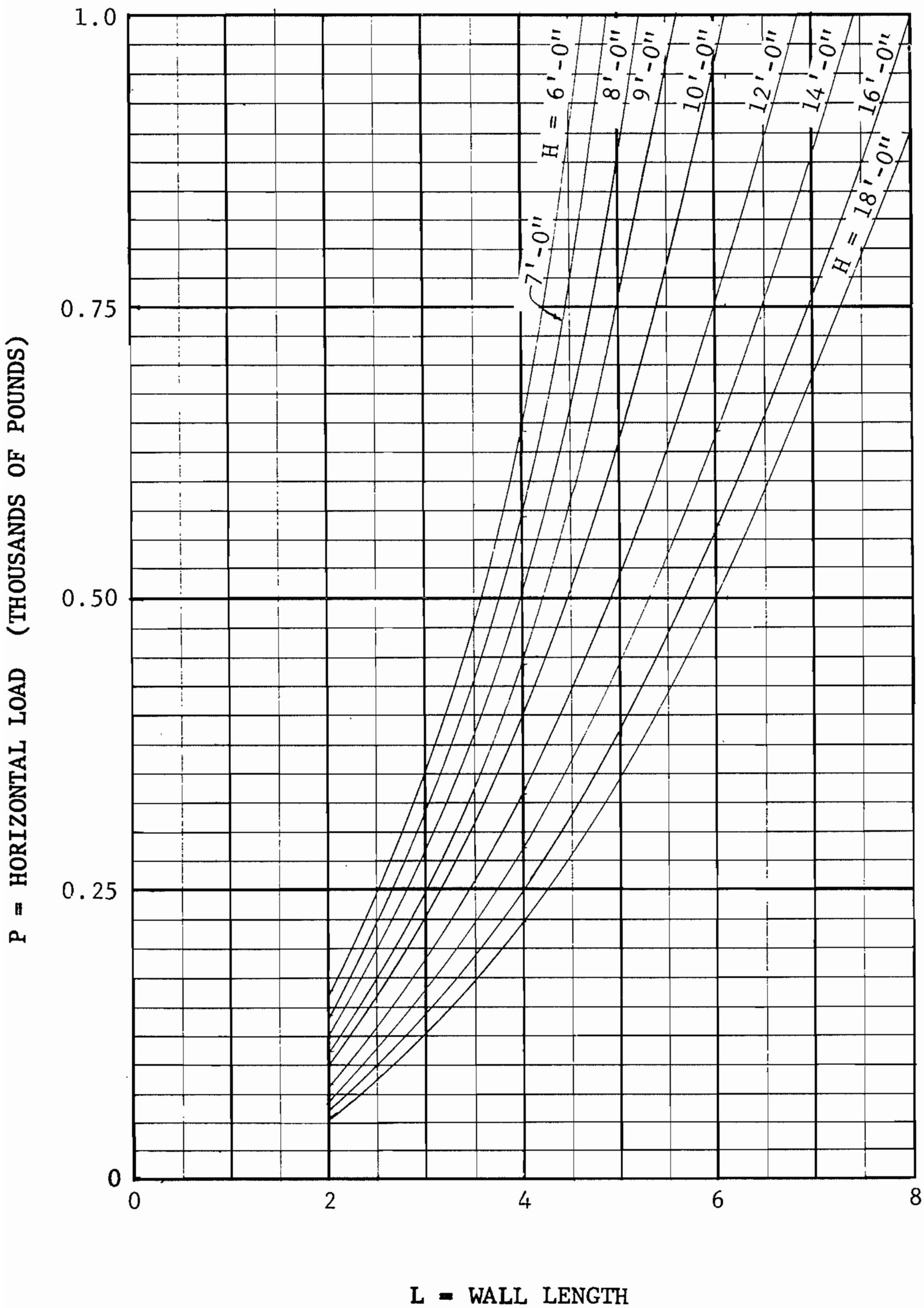
HOLD-DOWN GRAPH E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 400 lb/ft



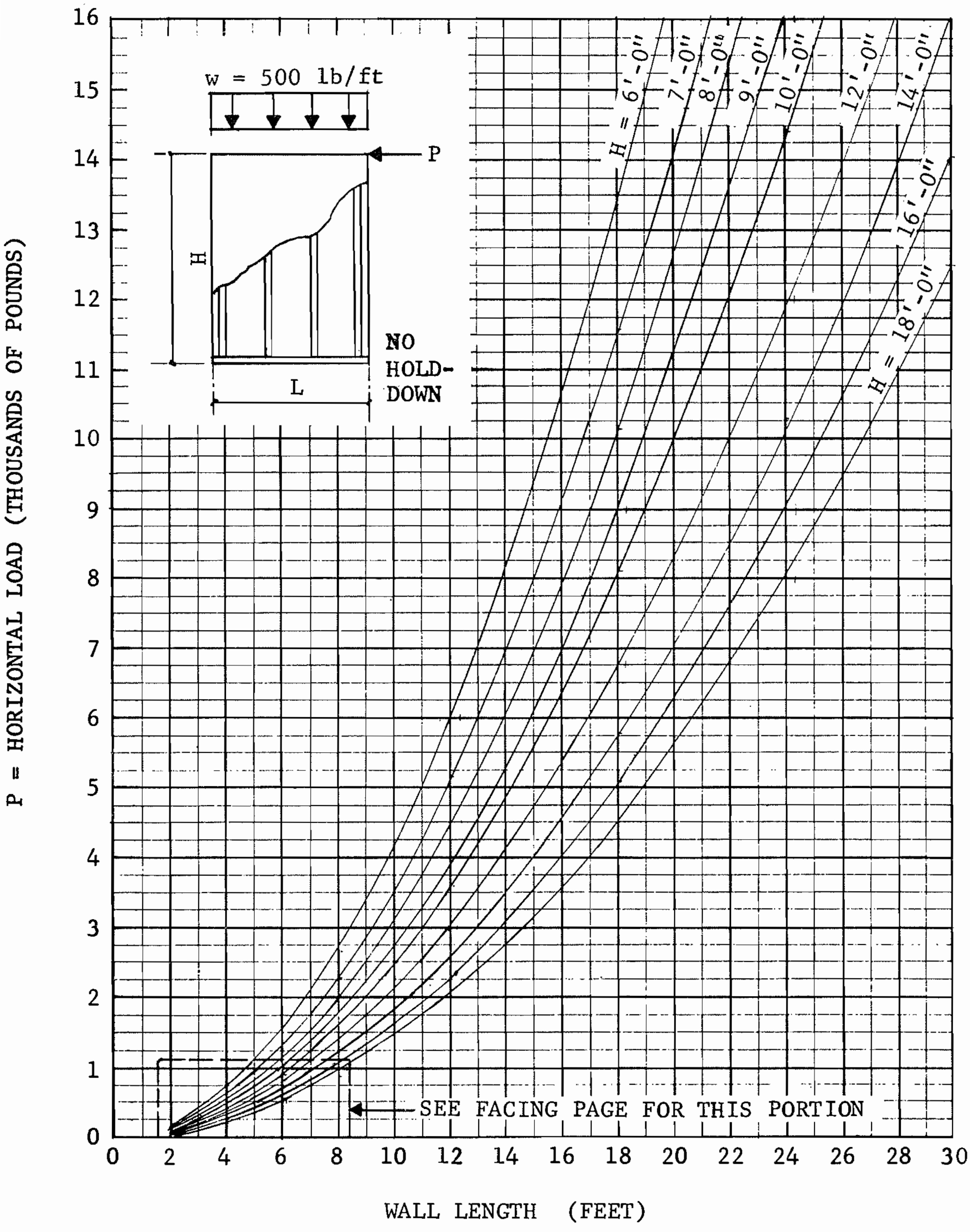
HOLD-DOWN GRAPH E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 400 lb/ft



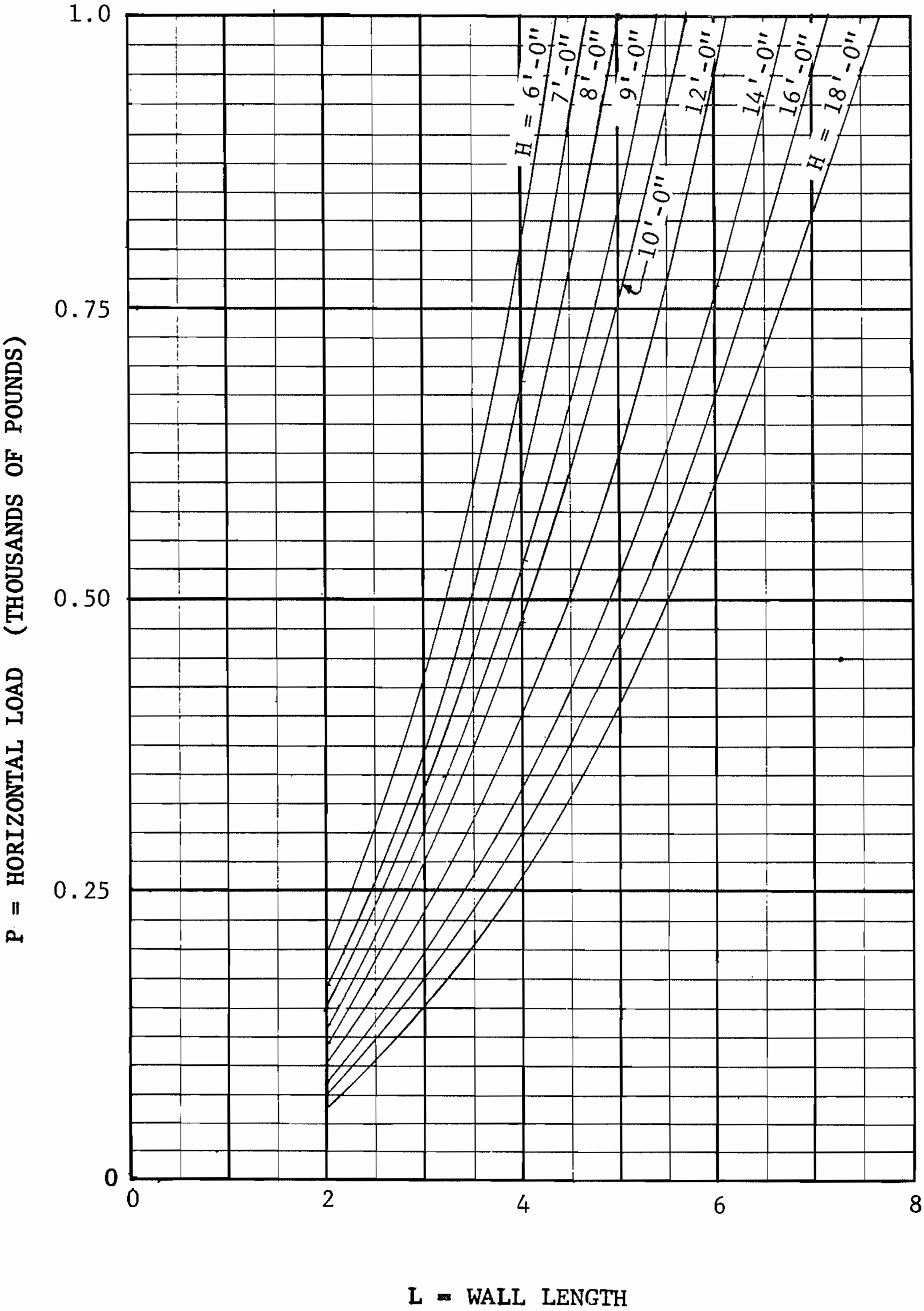
HOLD-DOWN GRAPH F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 500 lb/ft



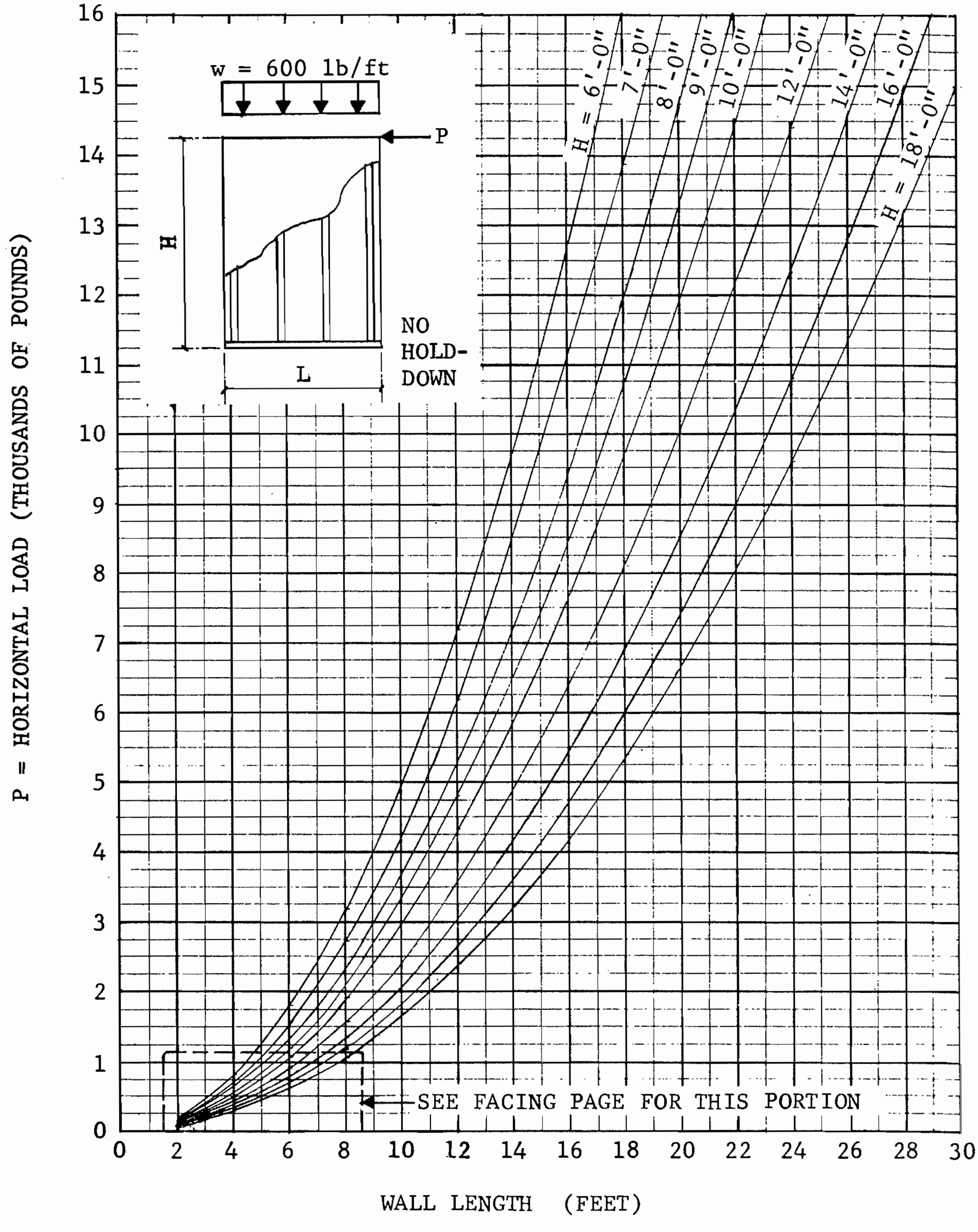
HOLD-DOWN GRAPH F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED — w = 500 lb/ft



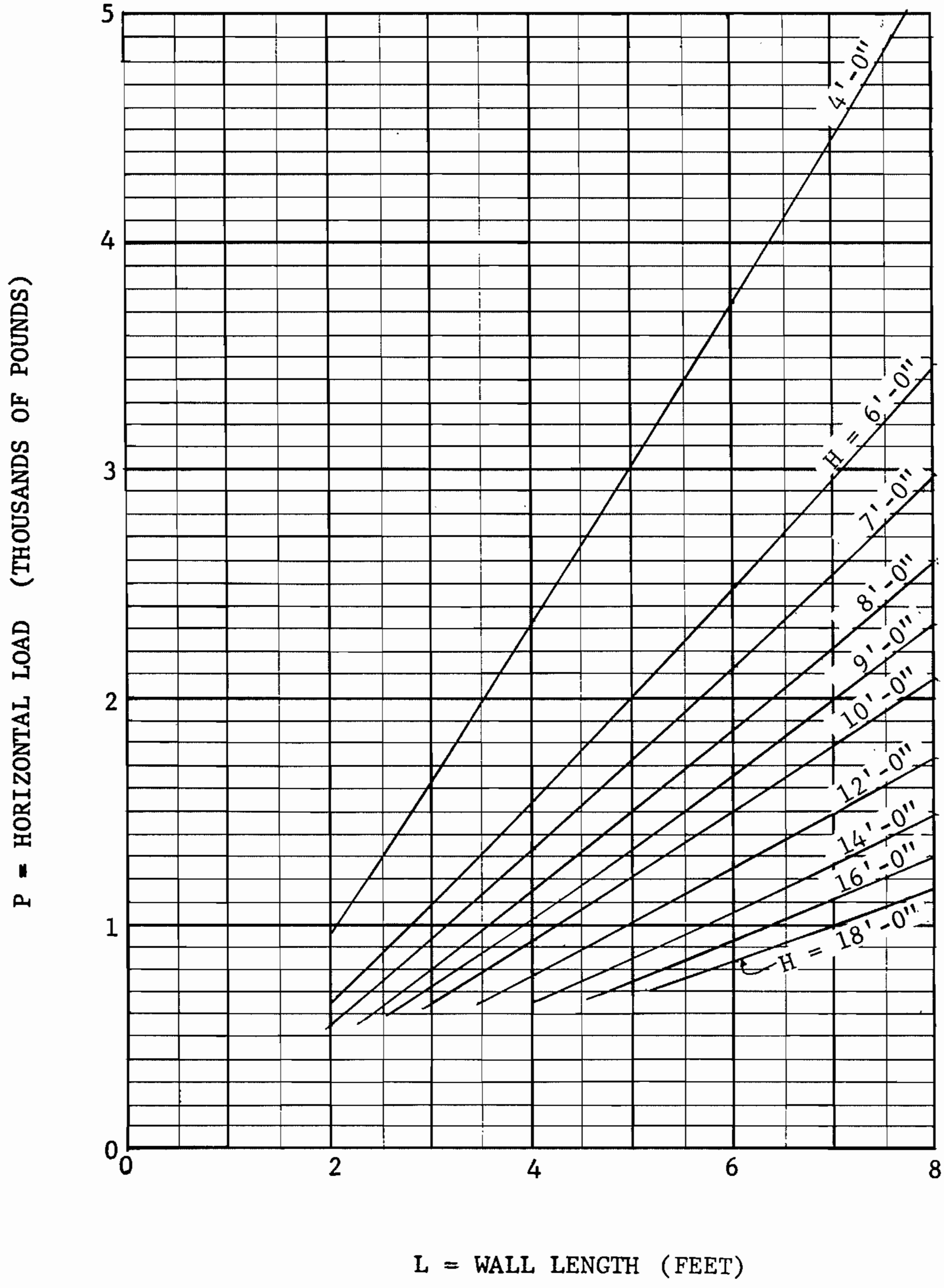
HOLD-DOWN GRAPH G

ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 600 lb/ft



HOLD-DOWN GRAPH G

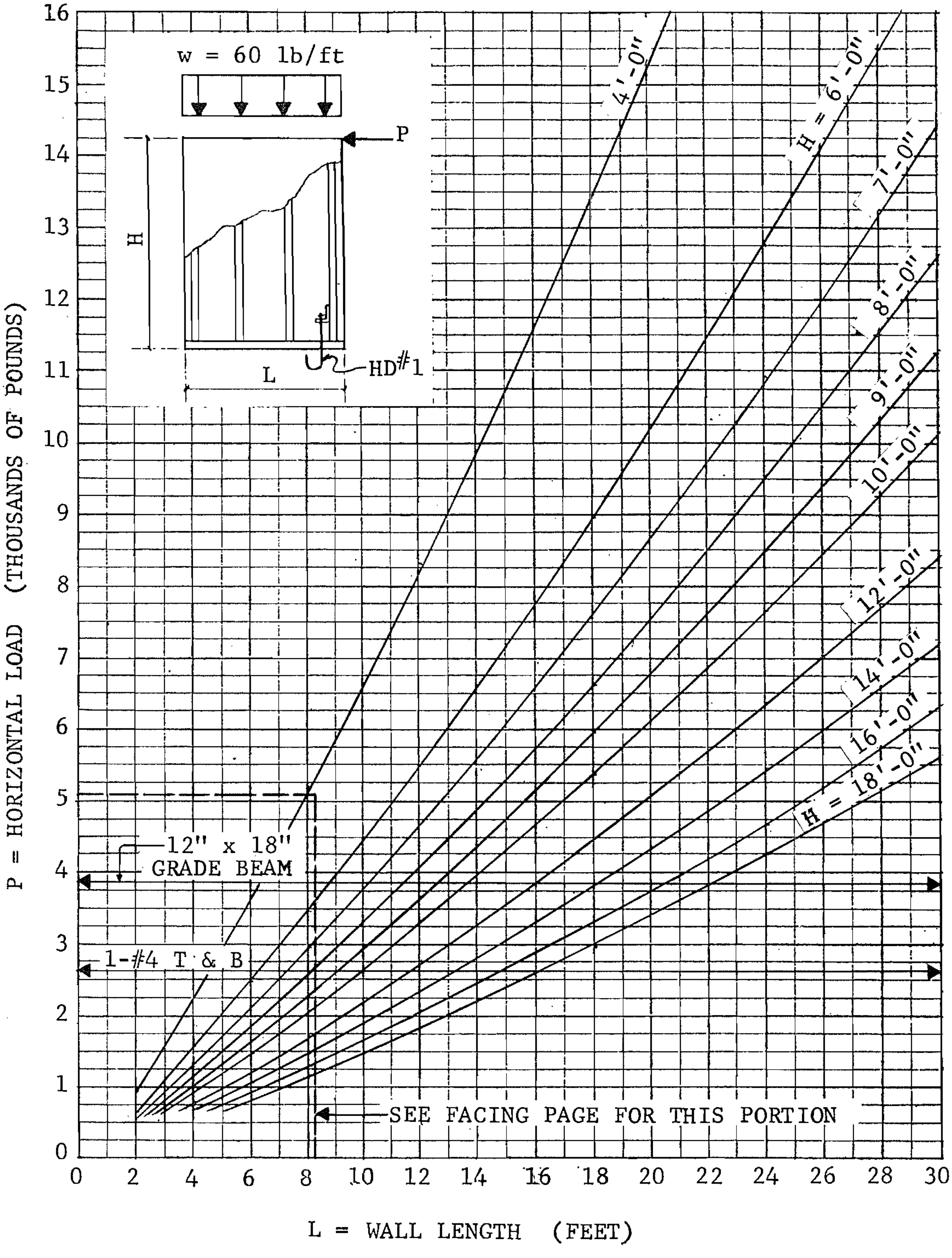
ALLOWABLE HORIZONTAL LOAD (P) FOR:
NO HOLD-DOWN REQUIRED – w = 600 lb/ft



HOLD-DOWN GRAPH 1.A

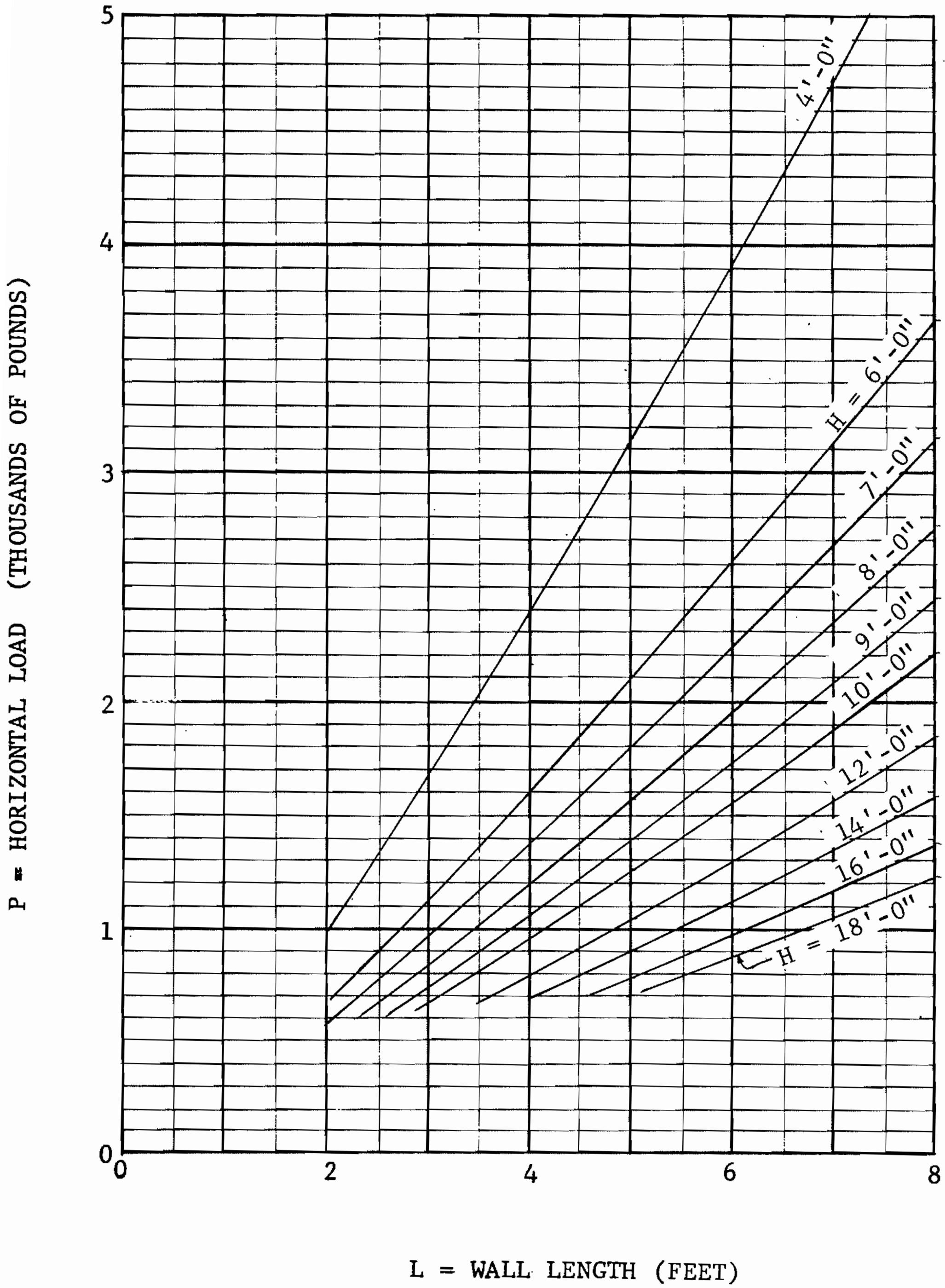
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 1 – w = 60 lb/ft



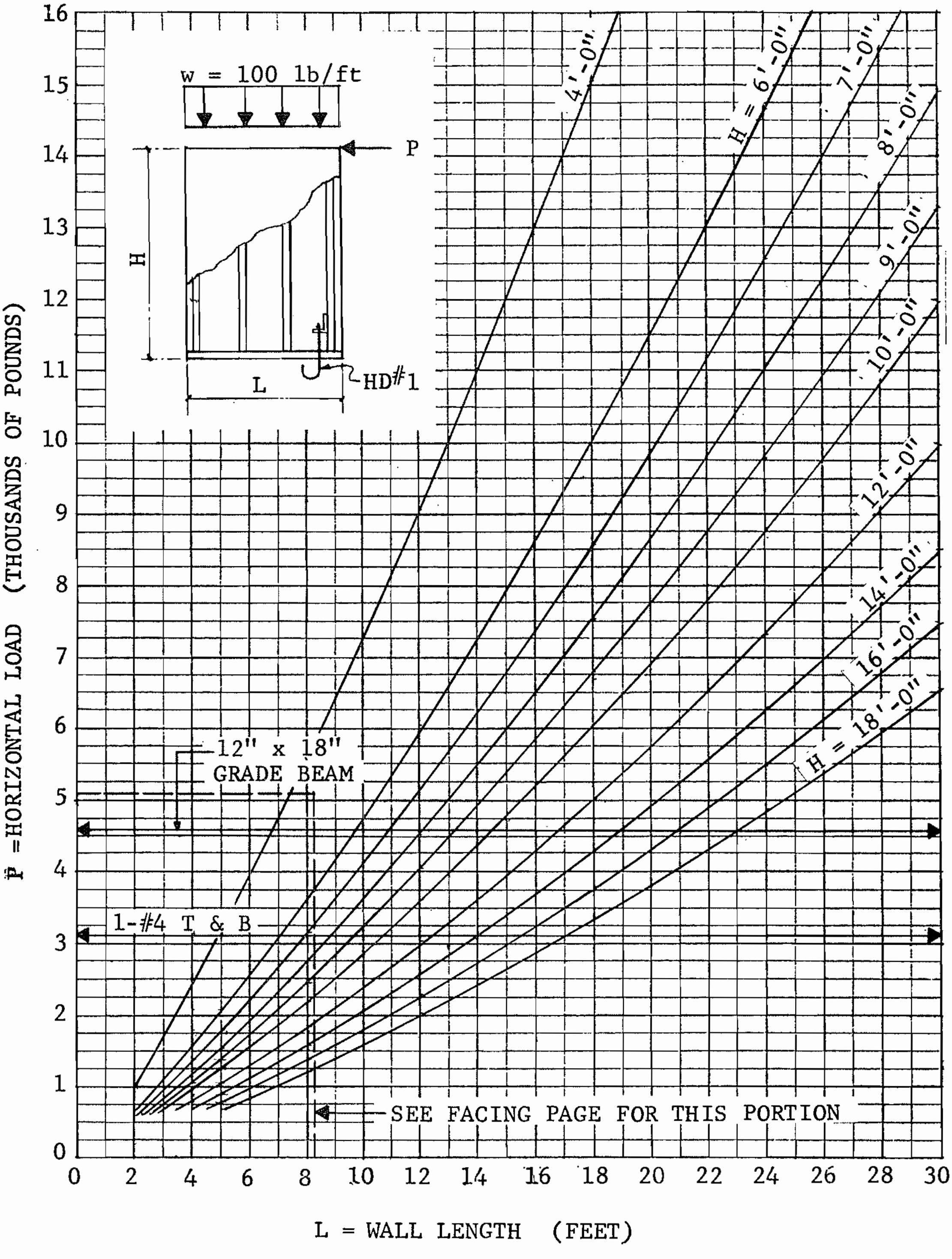
HOLD-DOWN GRAPH 1.A

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 – w = 60 lb/ft



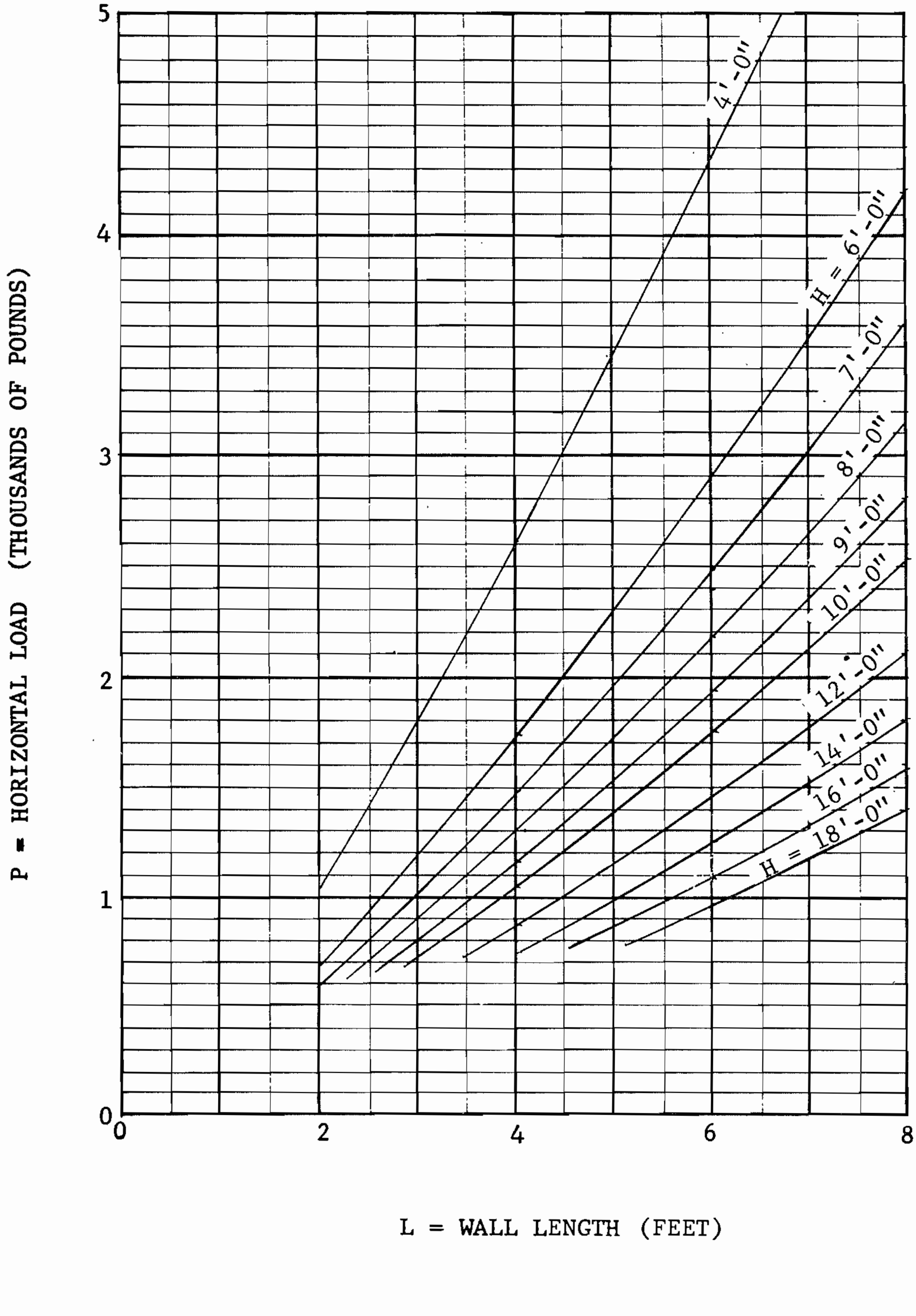
HOLD-DOWN GRAPH 1.B

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 – w = 100 lb/ft



HOLD-DOWN GRAPH 1.B

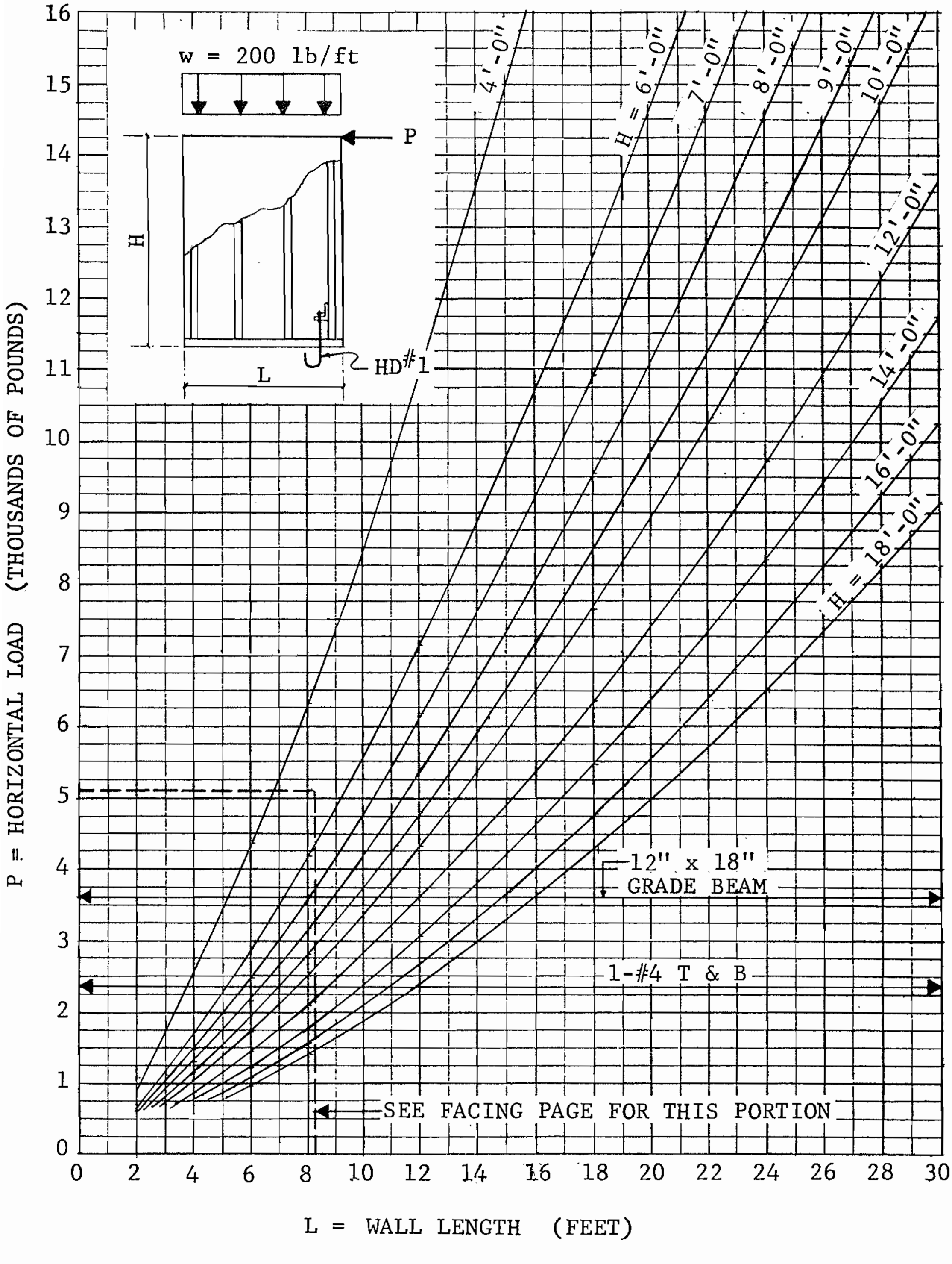
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 – w = 100 lb/ft



HOLD-DOWN GRAPH 1.C

ALLOWABLE HORIZONTAL LOAD (P) FOR:

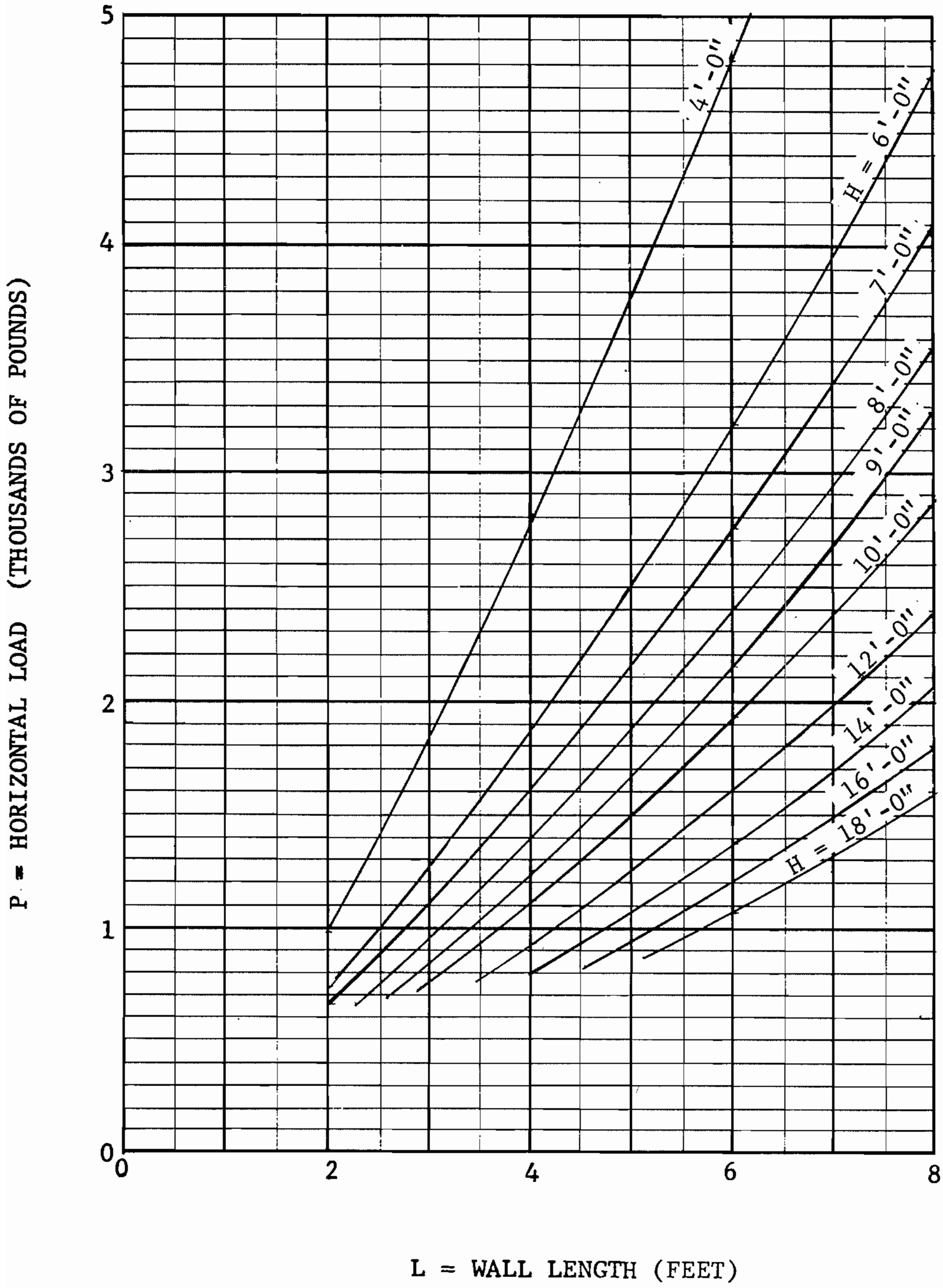
HOLD-DOWN # 1 – w = 200 lb/ft



HOLD-DOWN GRAPH 1.C

ALLOWABLE HORIZONTAL LOAD (P) FOR:

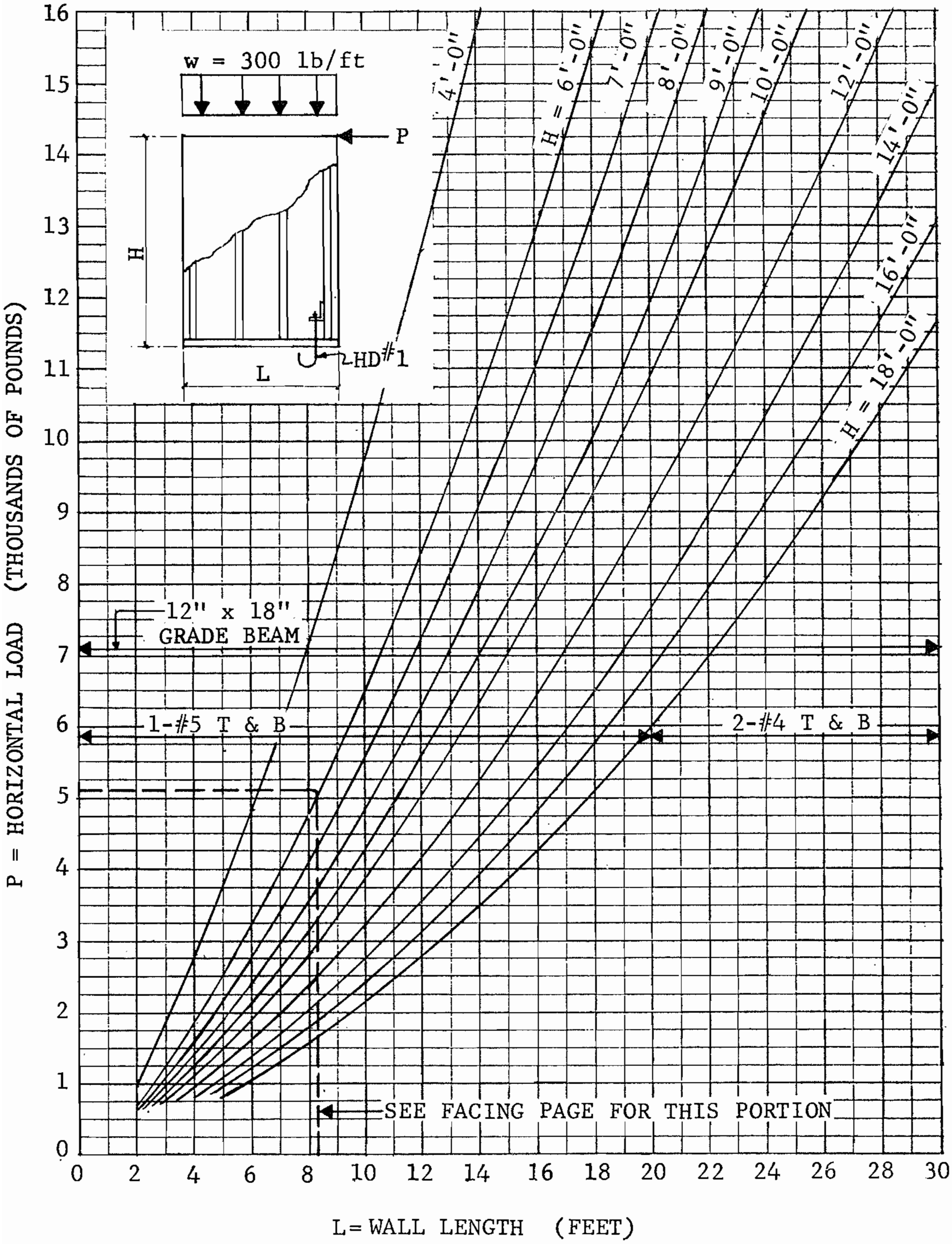
HOLD-DOWN # 1 — w = 200 lb/ft



HOLD-DOWN GRAPH 1.D

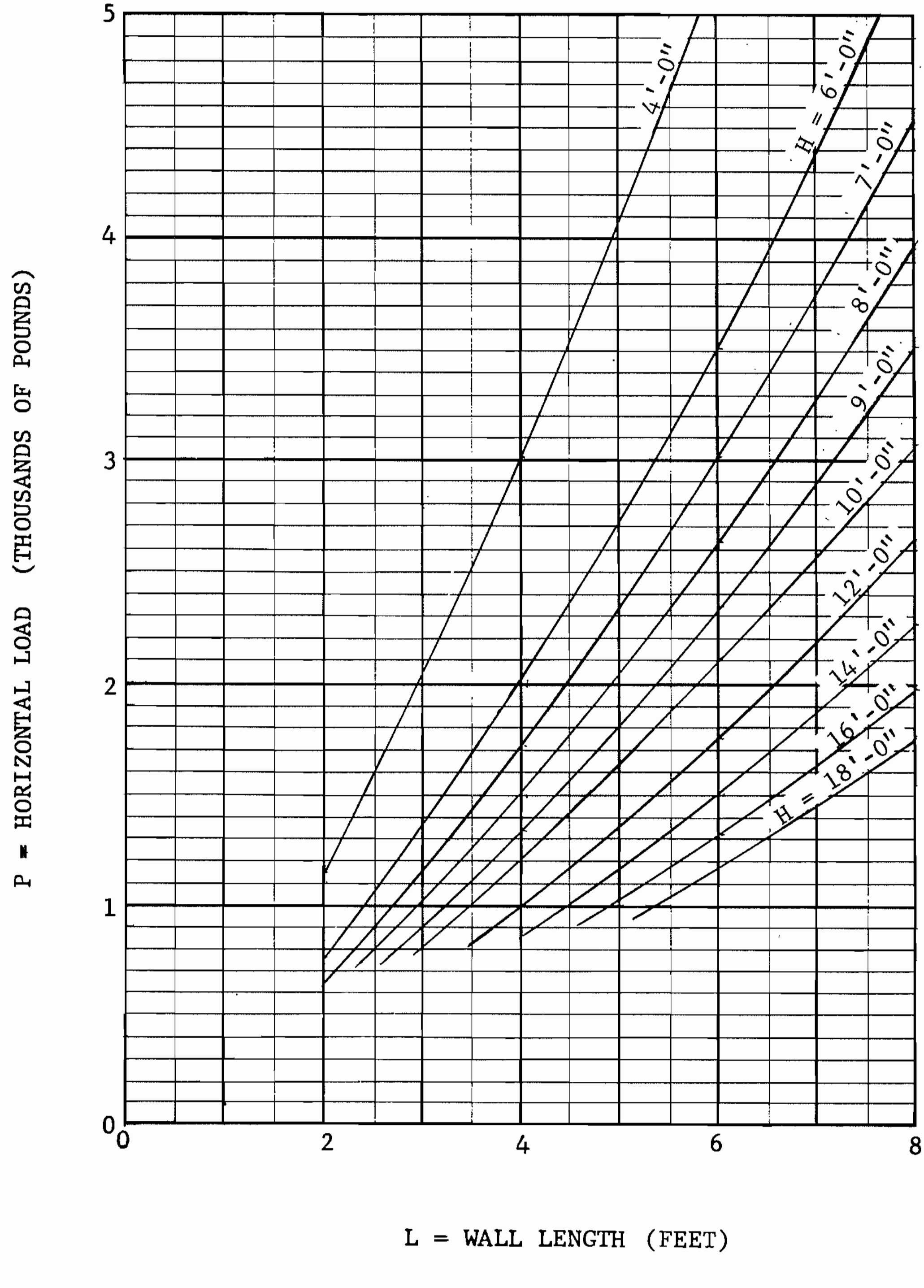
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 1 – w = 300 lb/ft



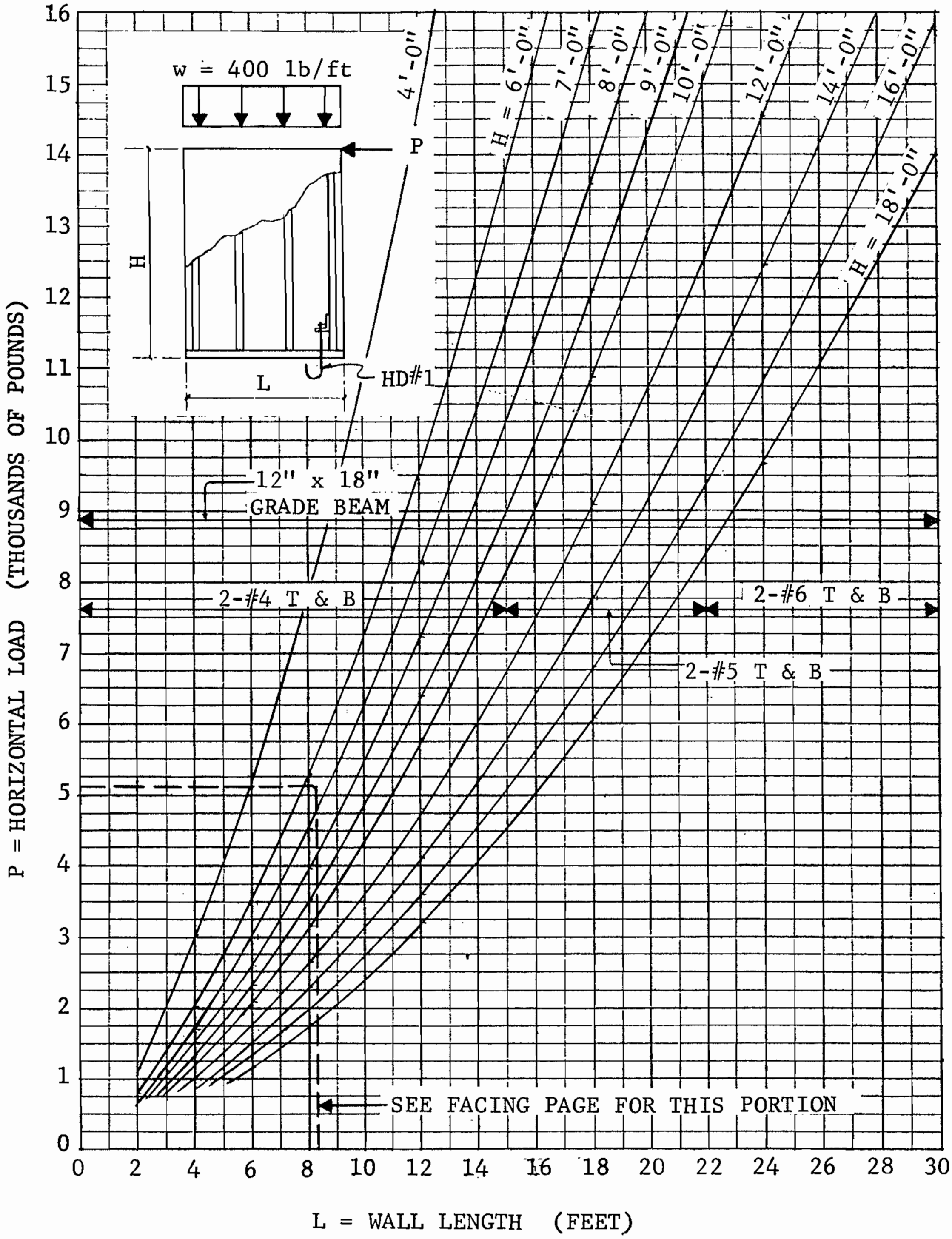
HOLD-DOWN GRAPH 1.D

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 — w = 300 lb/ft



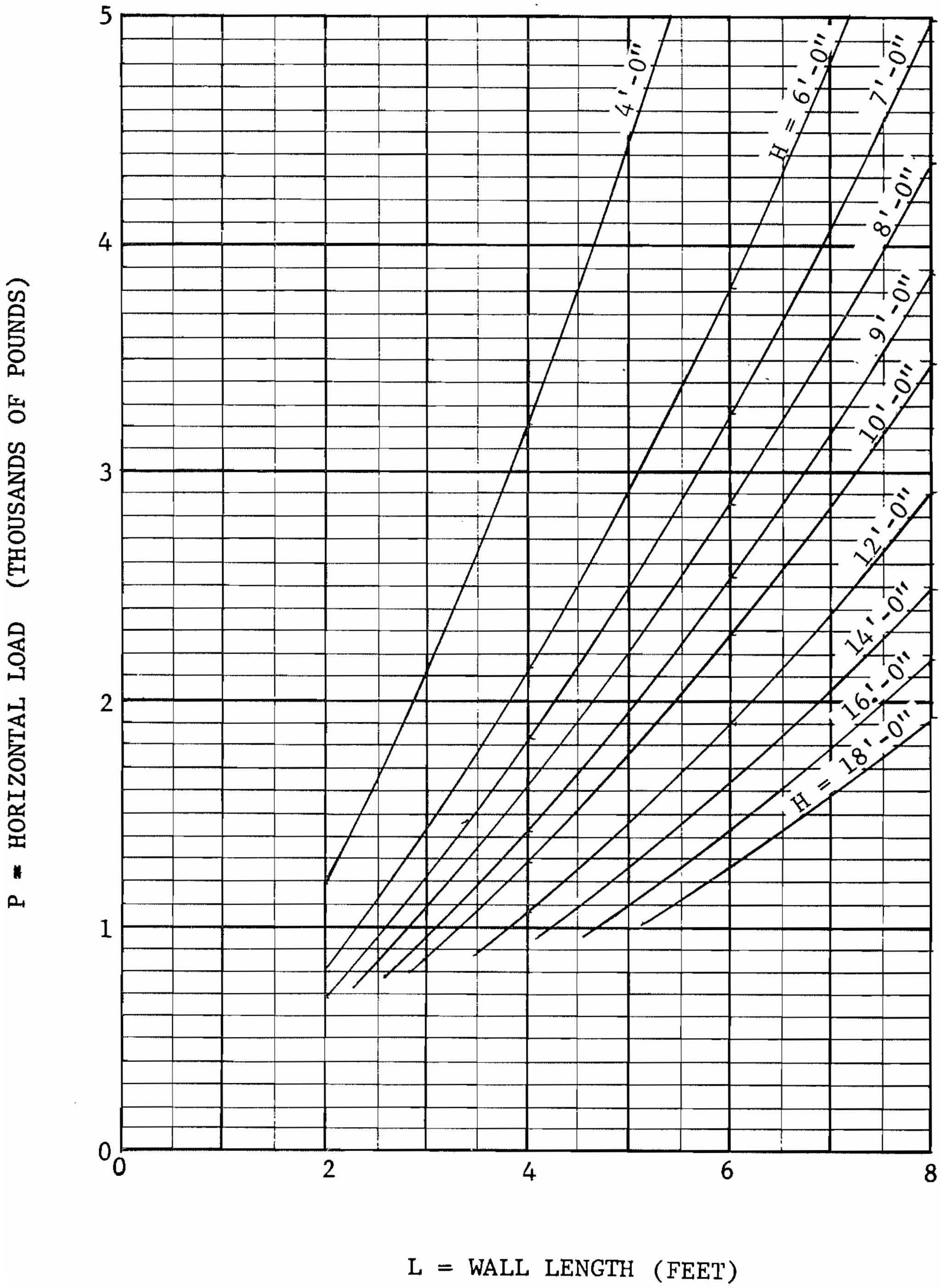
HOLD-DOWN GRAPH 1.E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 — w = 400 lb/ft



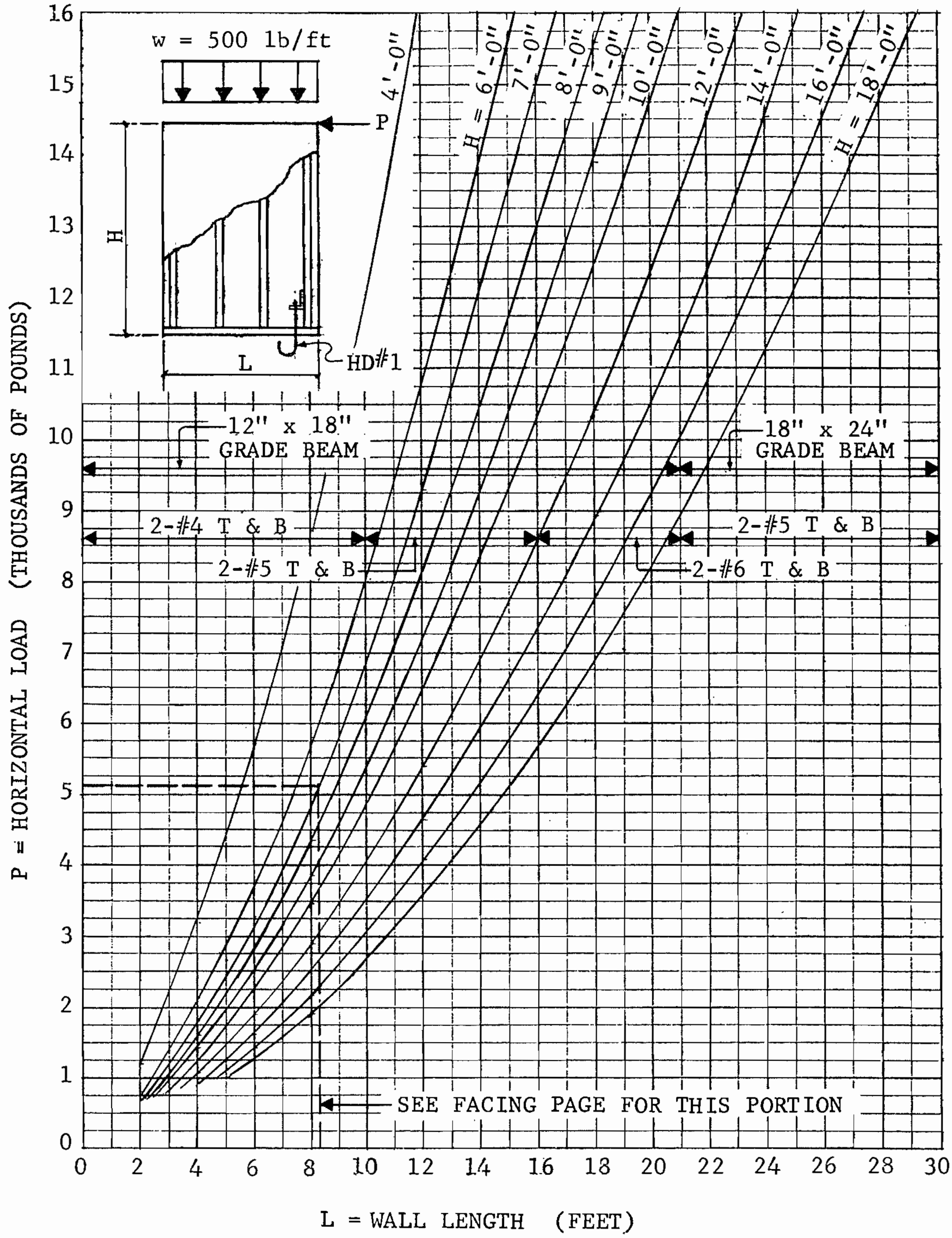
HOLD-DOWN GRAPH 1.E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 — w = 400 lb/ft



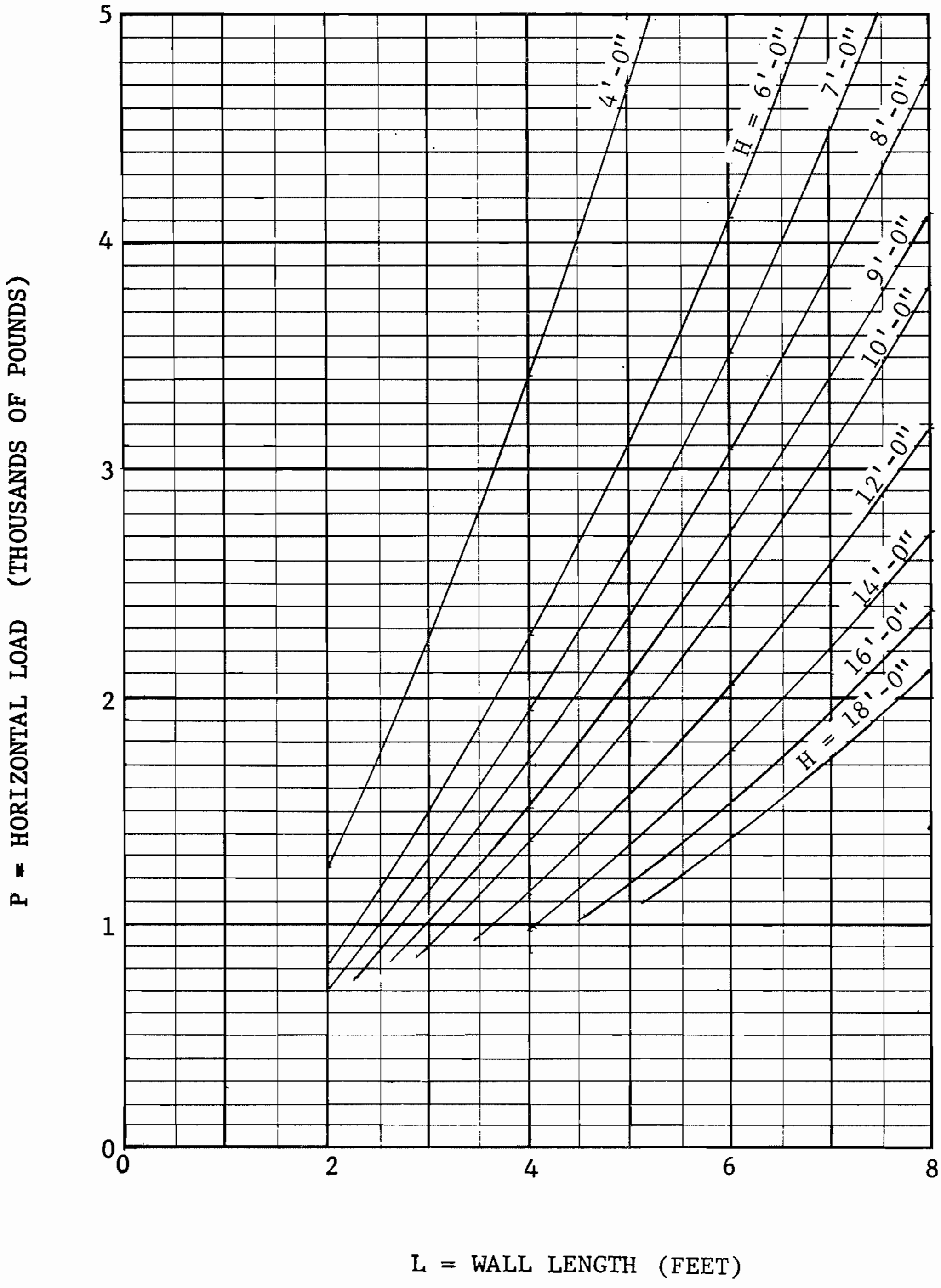
HOLD-DOWN GRAPH 1.F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 – w = 500 lb/ft



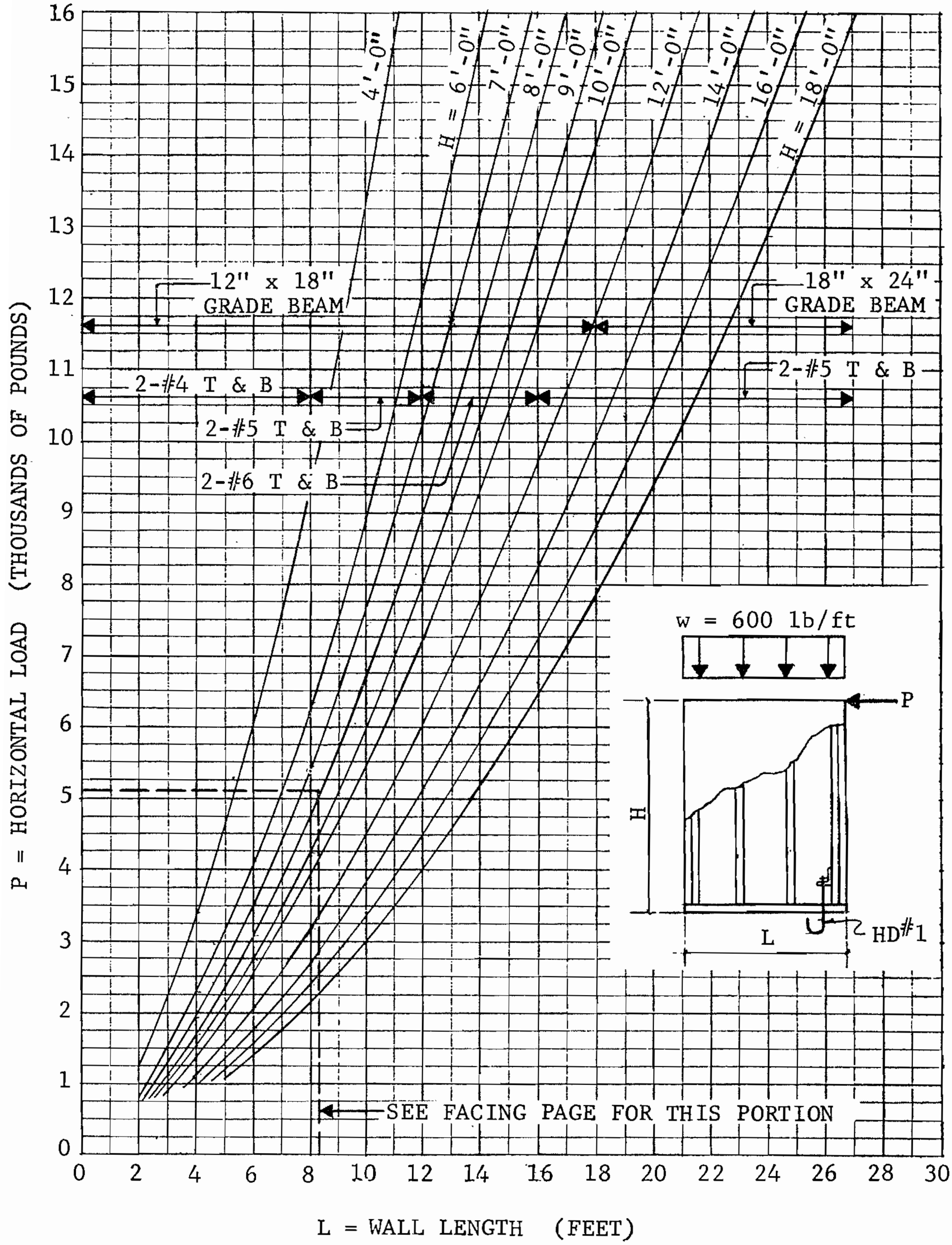
HOLD-DOWN GRAPH 1.F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 – w = 500 lb/ft



HOLD-DOWN GRAPH 1.G

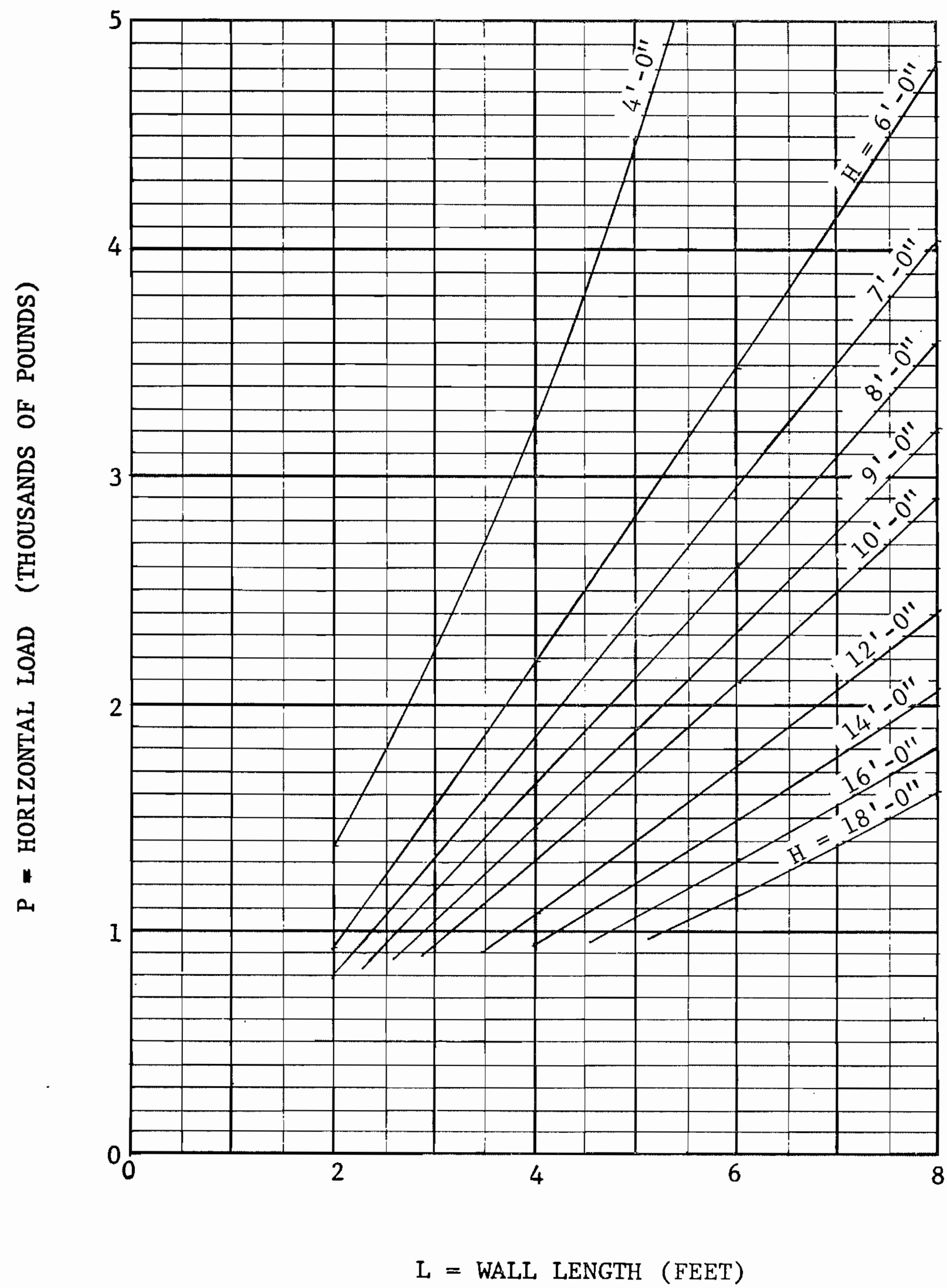
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 1 – w = 600 lb/ft



HOLD-DOWN GRAPH 1.G

ALLOWABLE HORIZONTAL LOAD (P) FOR:

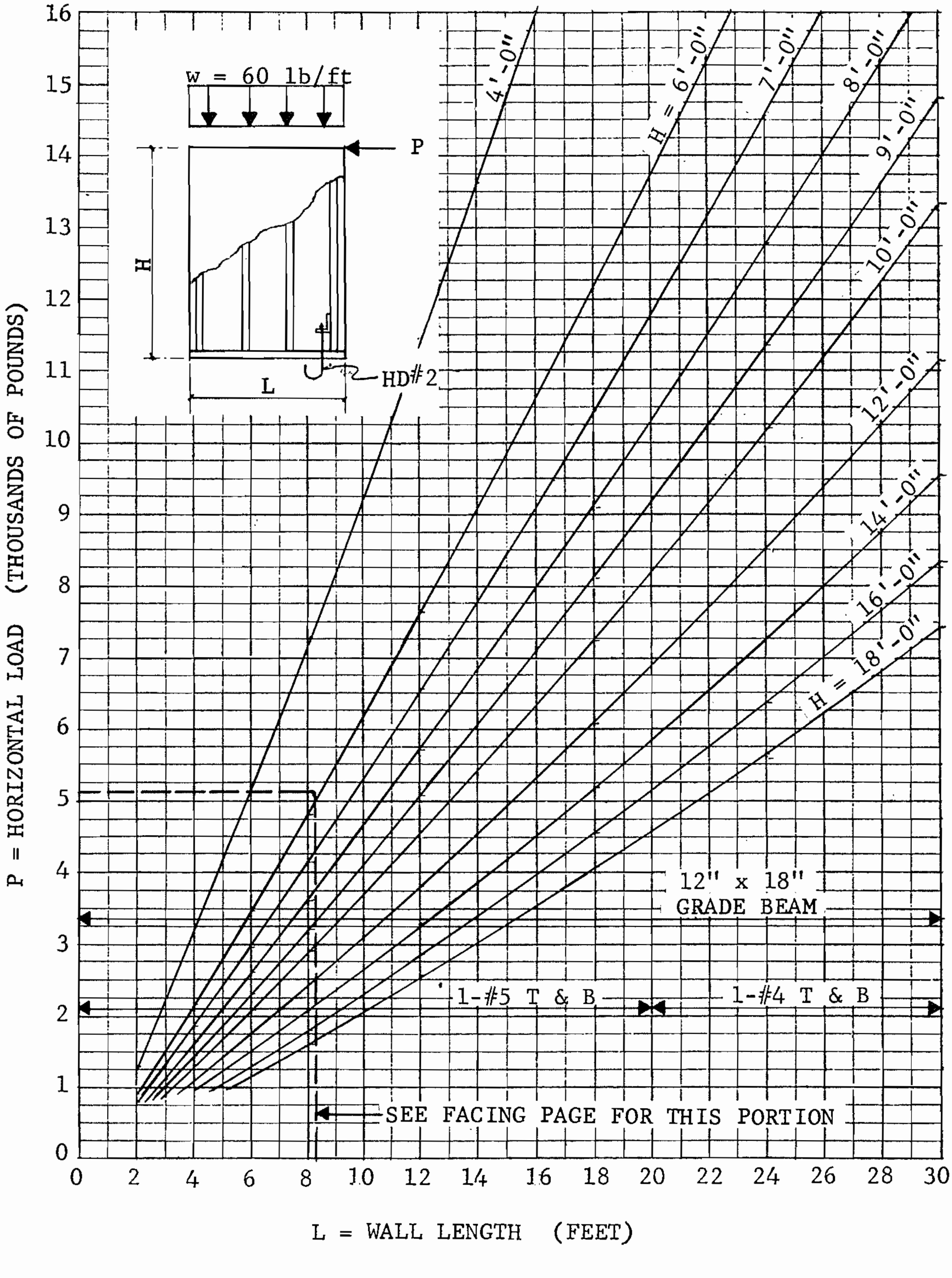
HOLD-DOWN # 1 – w = 600 lb/ft



HOLD-DOWN GRAPH 2.A

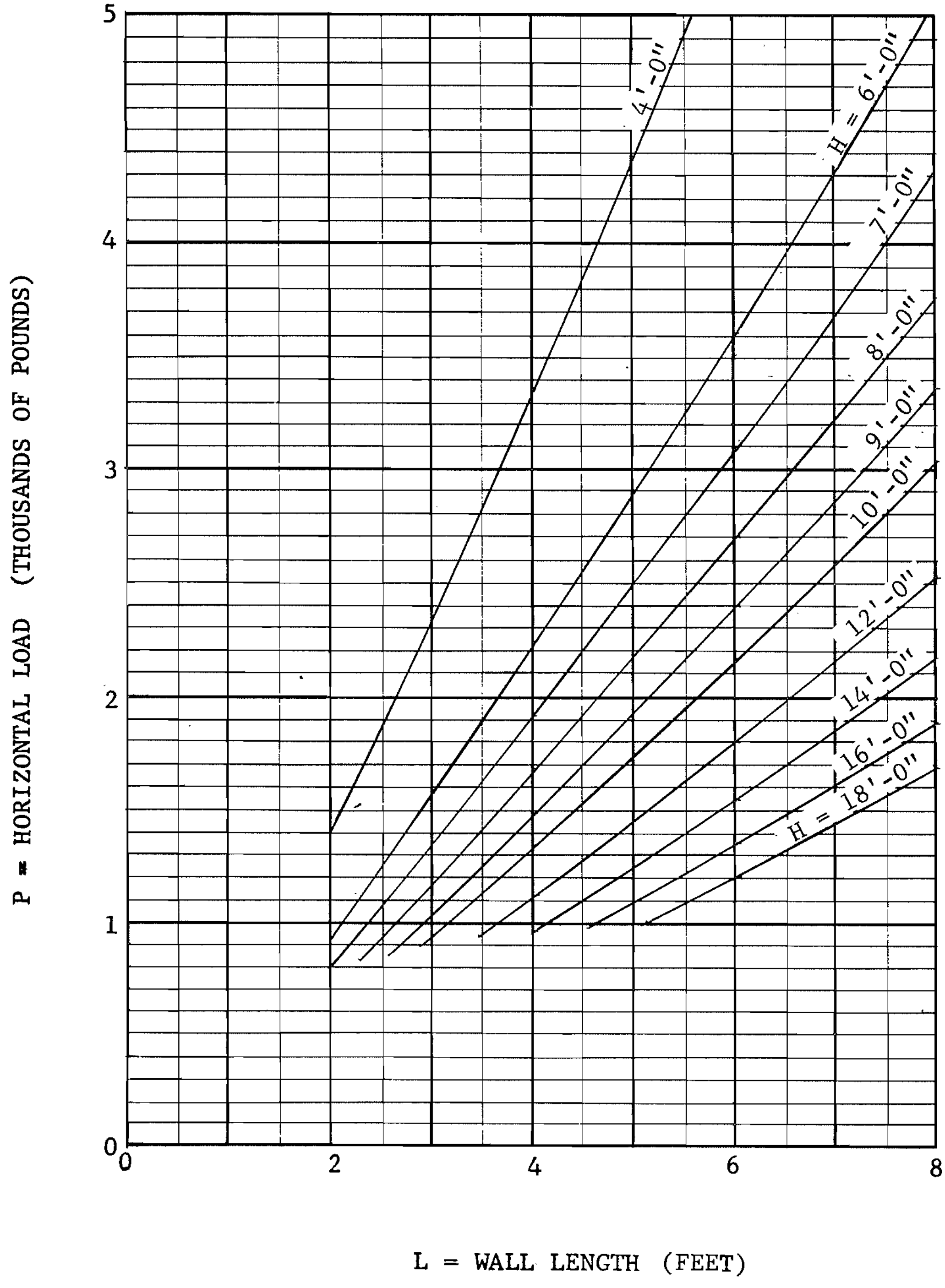
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 2 – w = 60 lb/ft



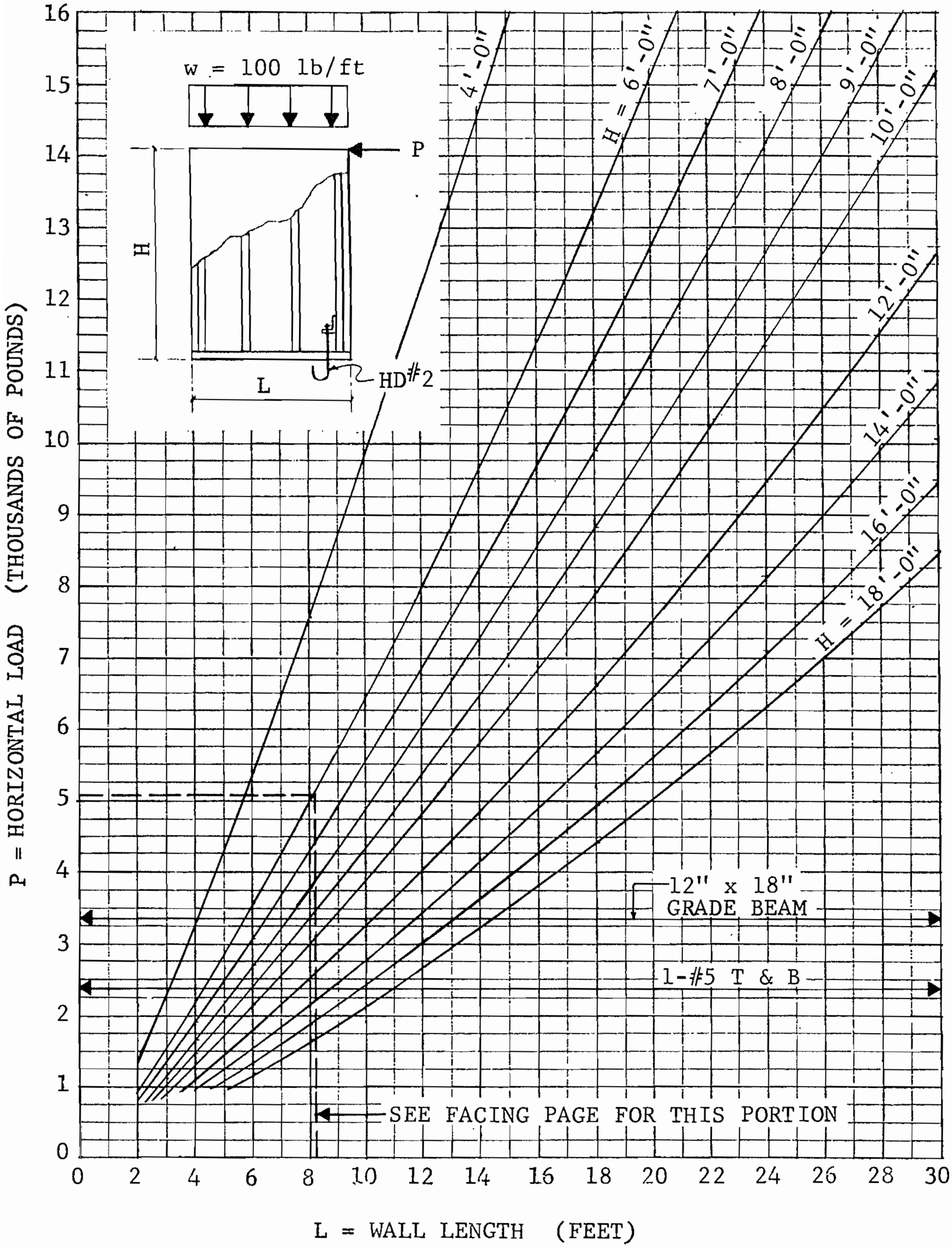
HOLD-DOWN GRAPH 2.A

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 60 lb/ft



HOLD-DOWN GRAPH 2.B

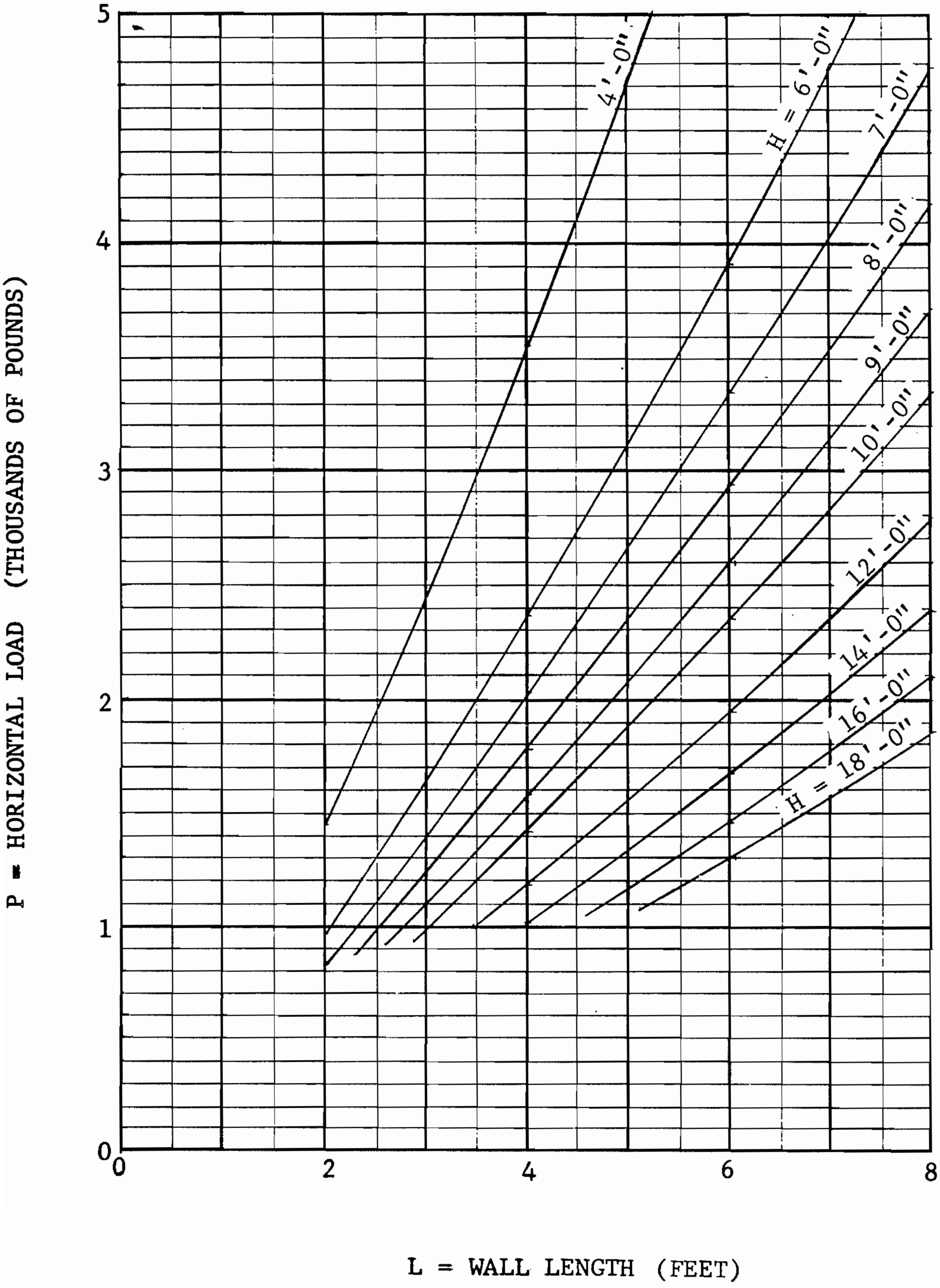
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 — w = 100 lb/ft



HOLD-DOWN GRAPH 2.B

ALLOWABLE HORIZONTAL LOAD (P) FOR:

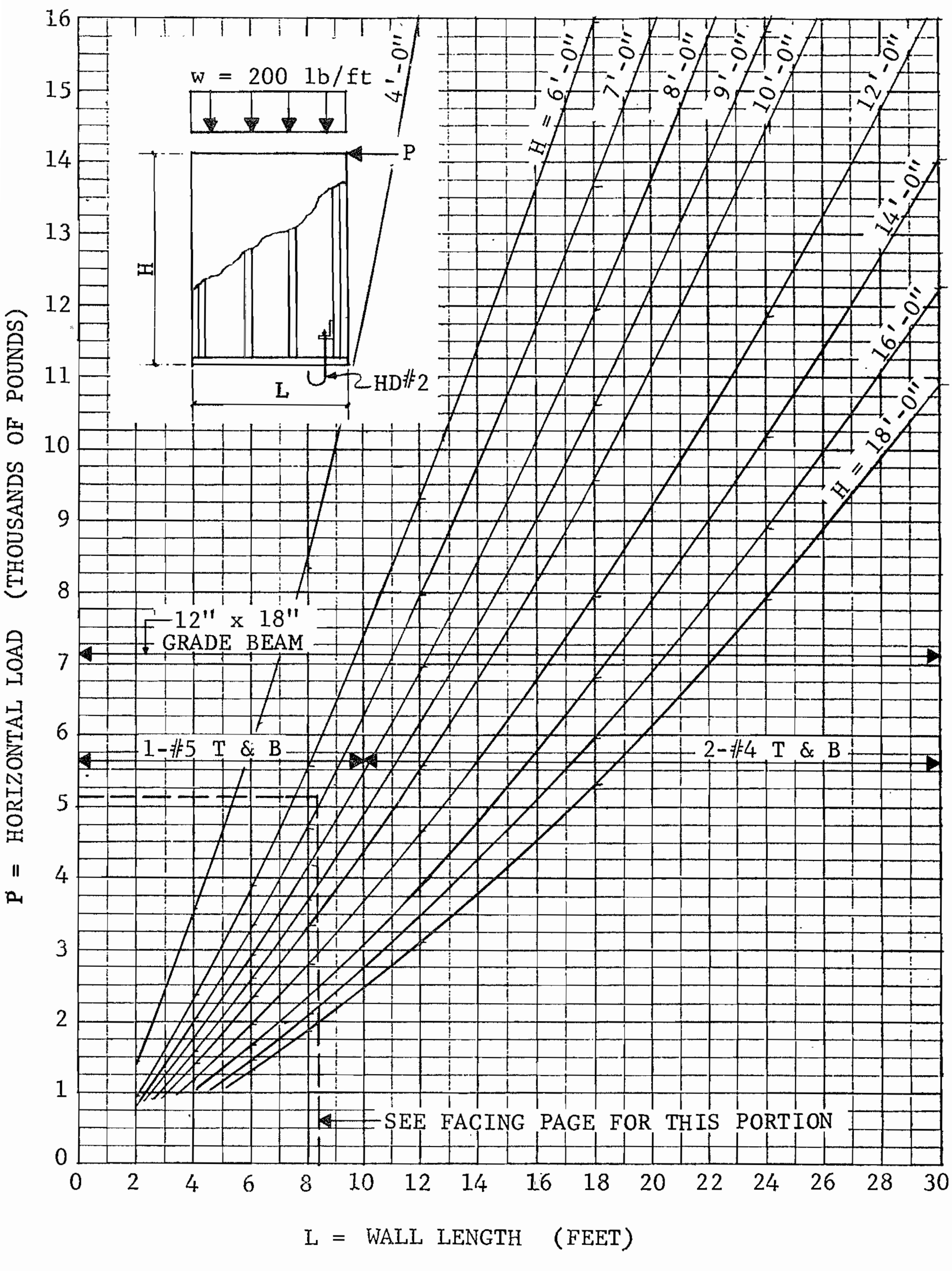
HOLD-DOWN # 2 – w = 100 lb/ft



HOLD-DOWN GRAPH 2.C

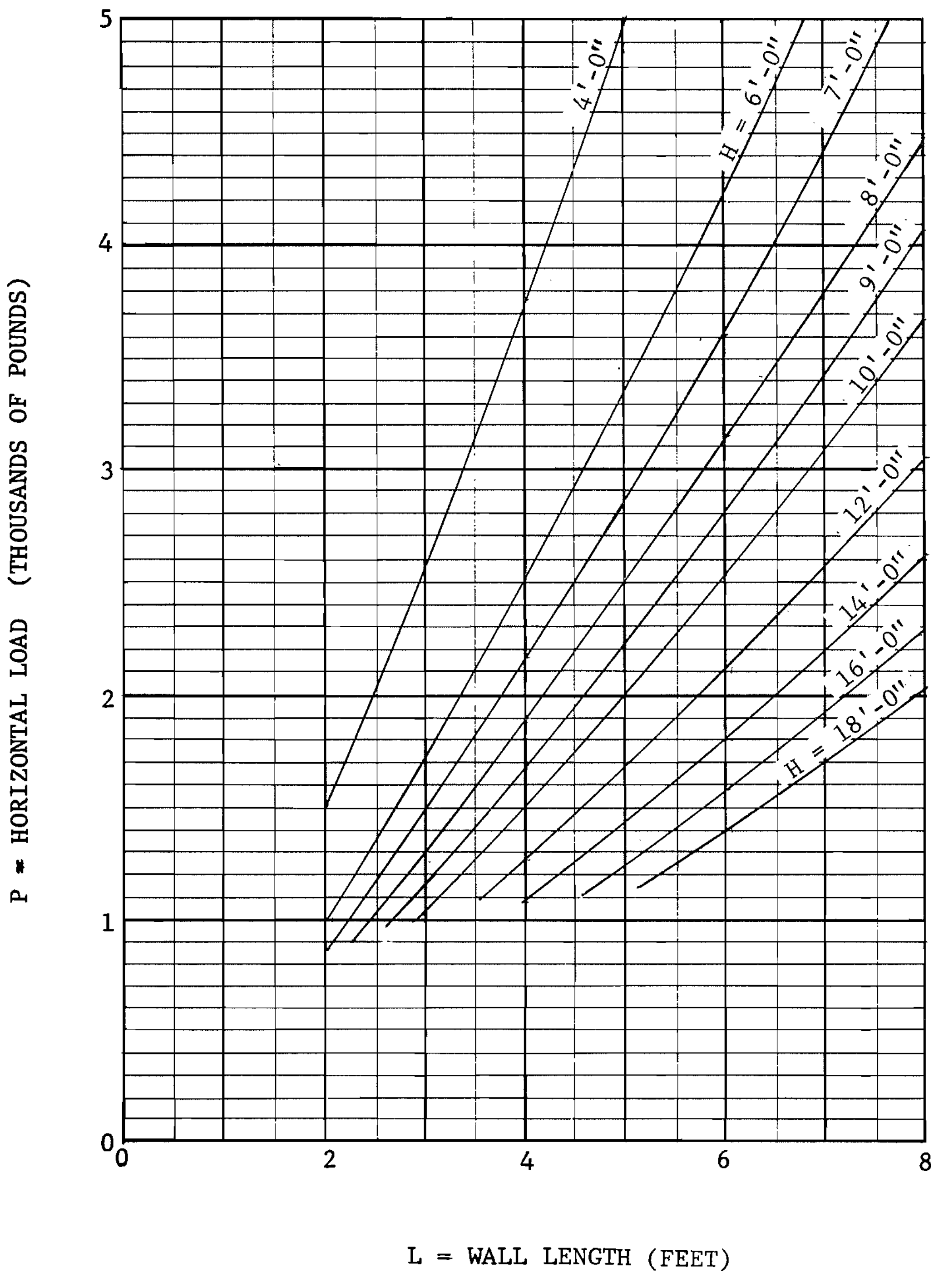
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 2 – w = 200 lb/ft



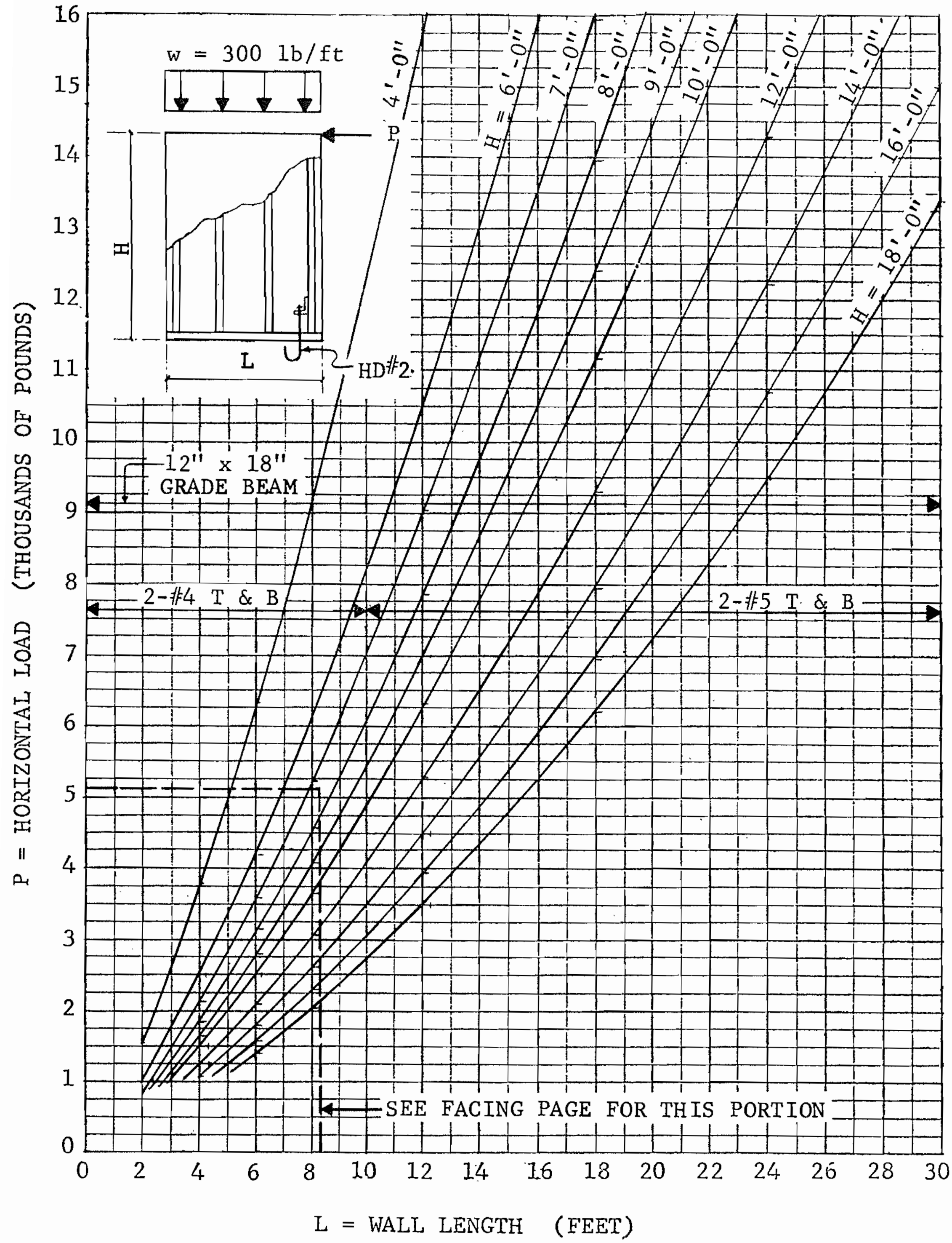
HOLD-DOWN GRAPH 2.C

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 200 lb/ft



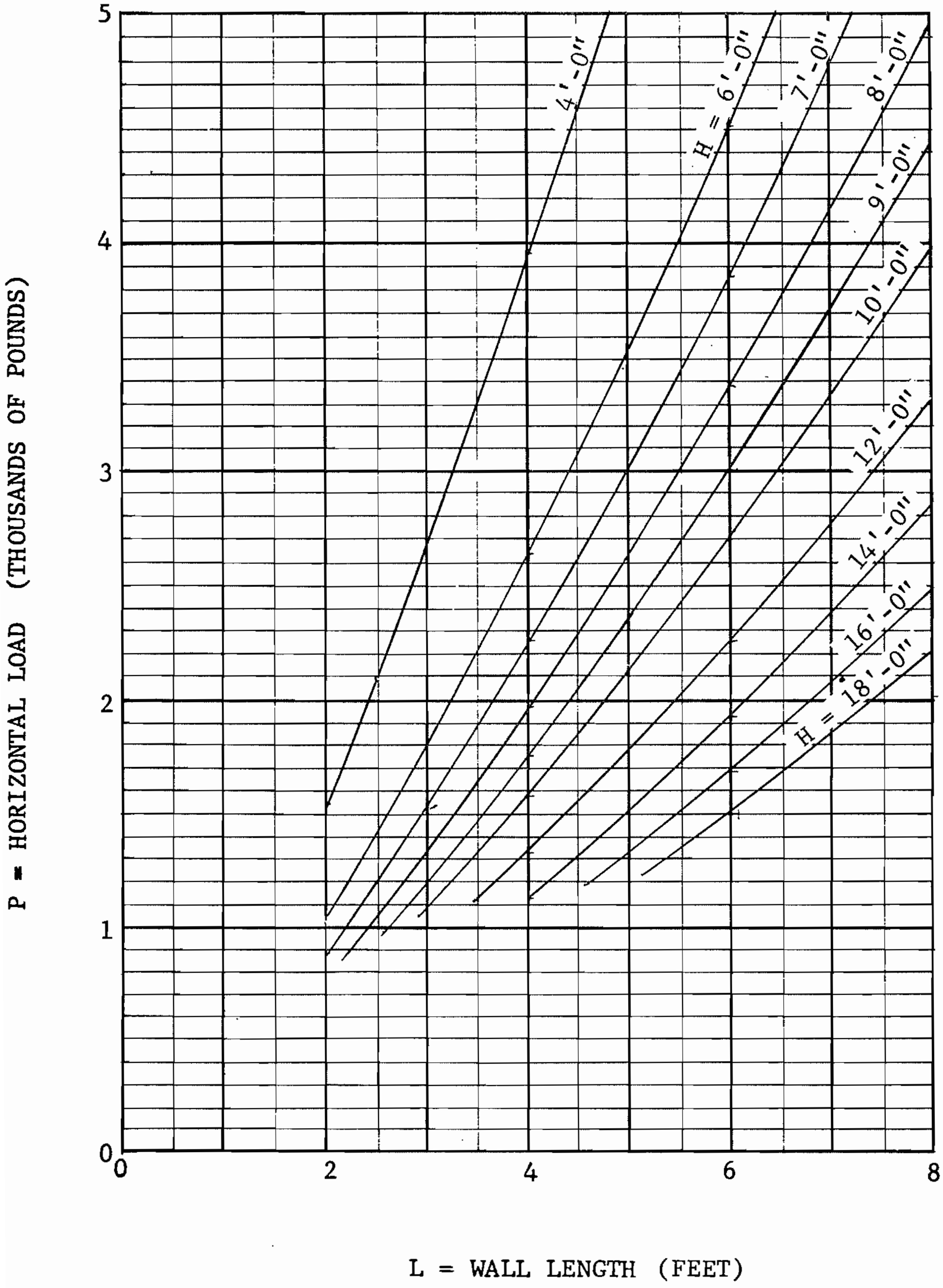
HOLD-DOWN GRAPH 2.D

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 300 lb/ft



HOLD-DOWN GRAPH 2.D

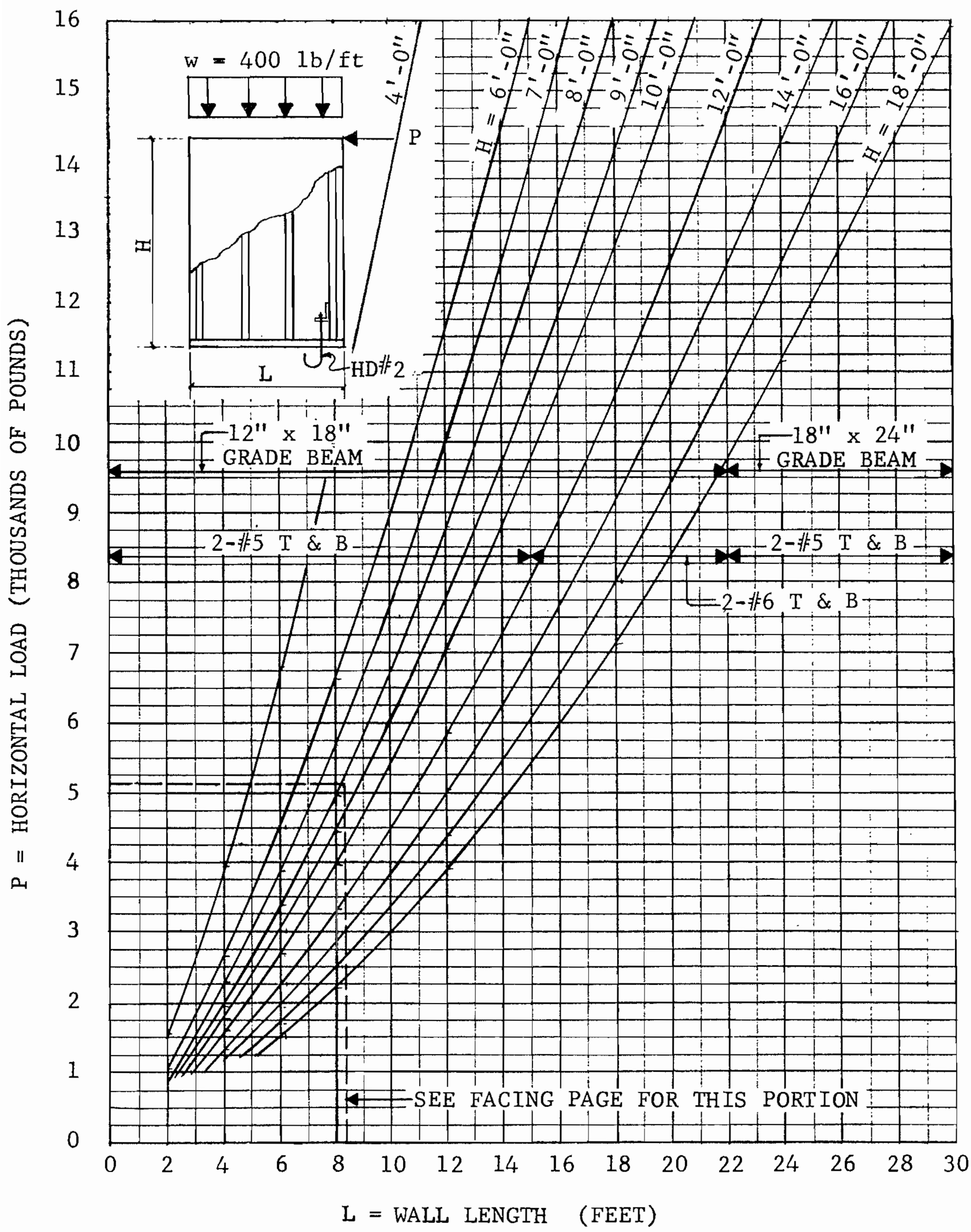
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 — w = 300 lb/ft



HOLD-DOWN GRAPH 2.E

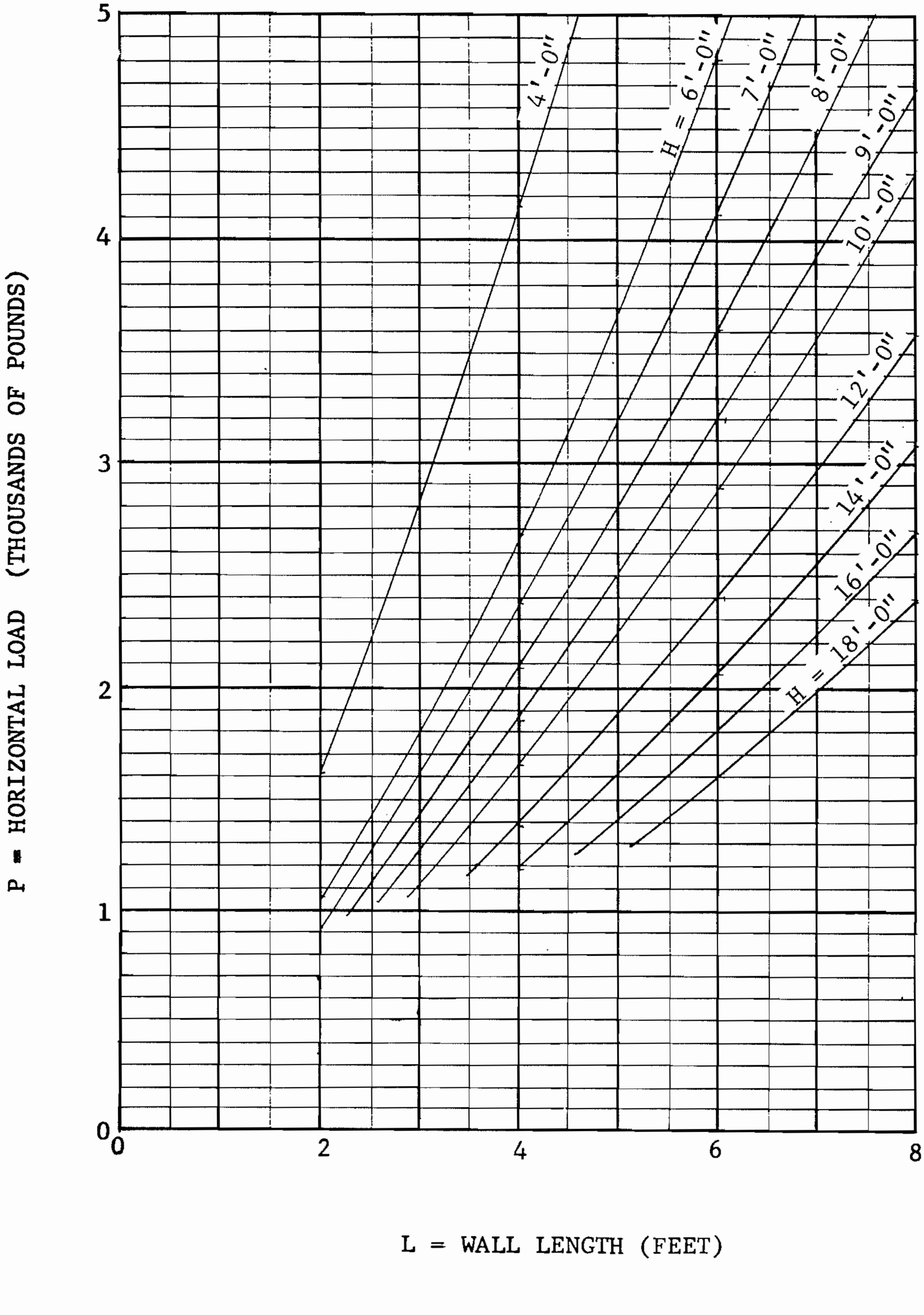
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 2 – w = 400 lb/ft



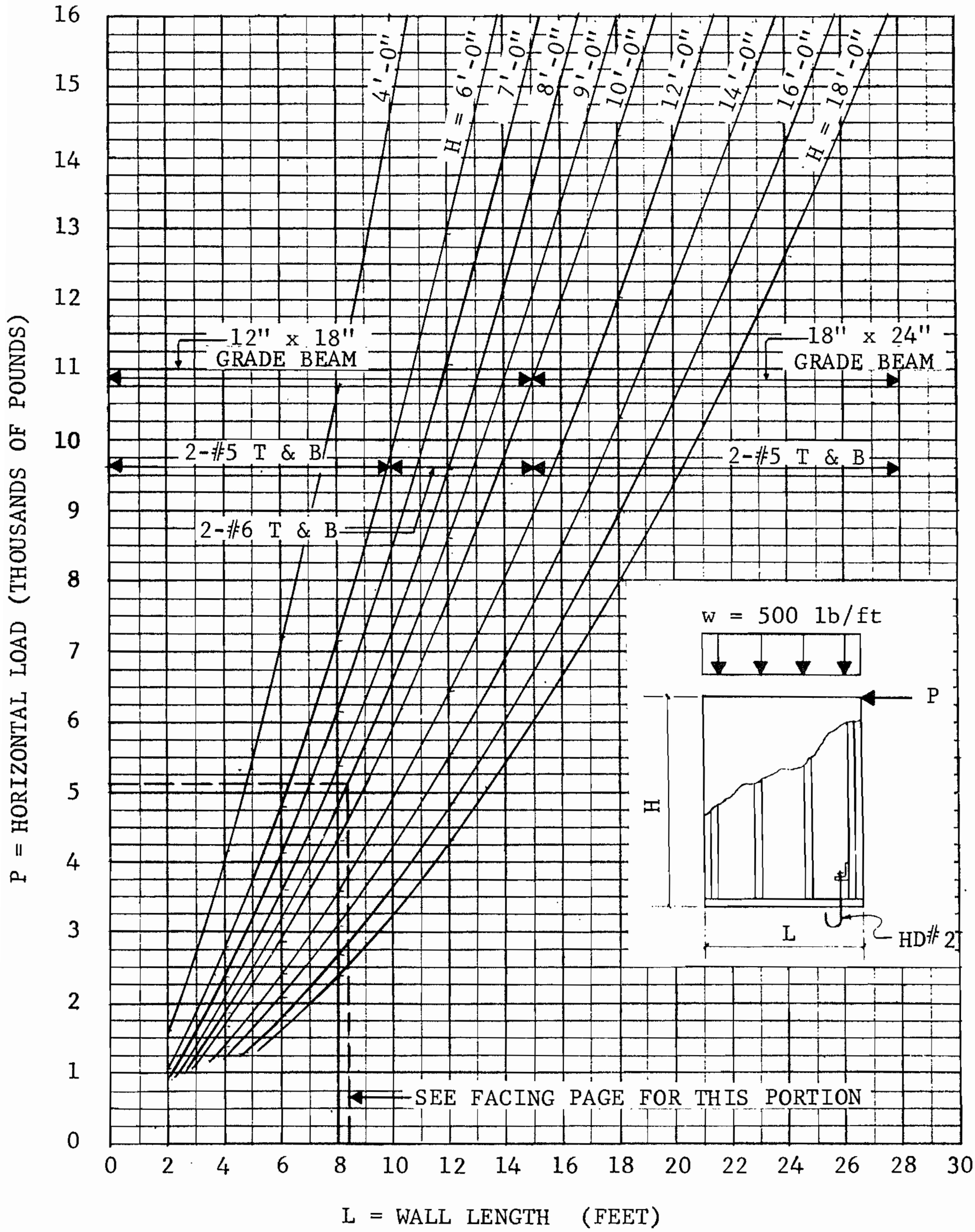
HOLD-DOWN GRAPH 2.E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 400 lb/ft



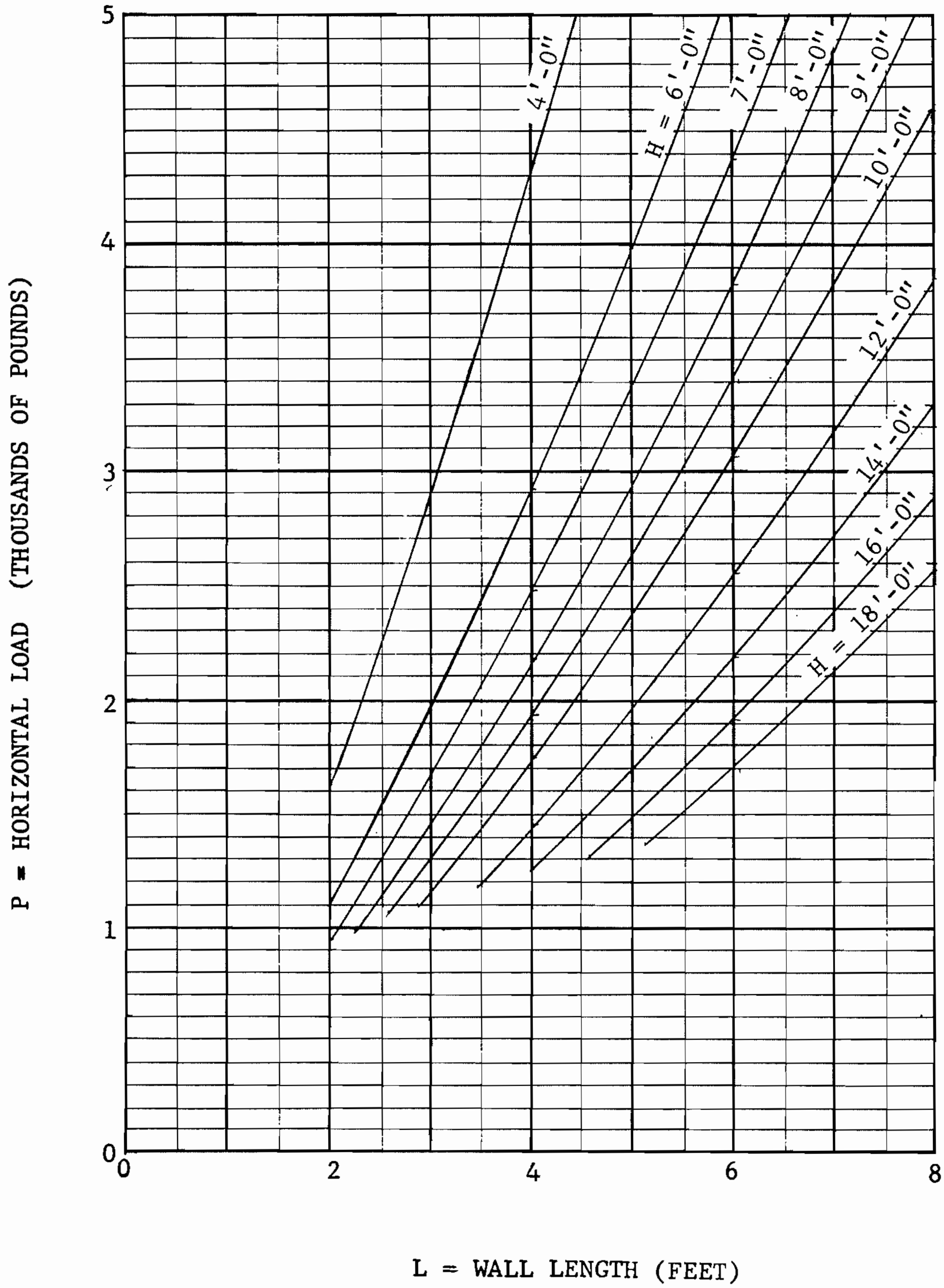
HOLD-DOWN GRAPH 2.F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 500 lb/ft



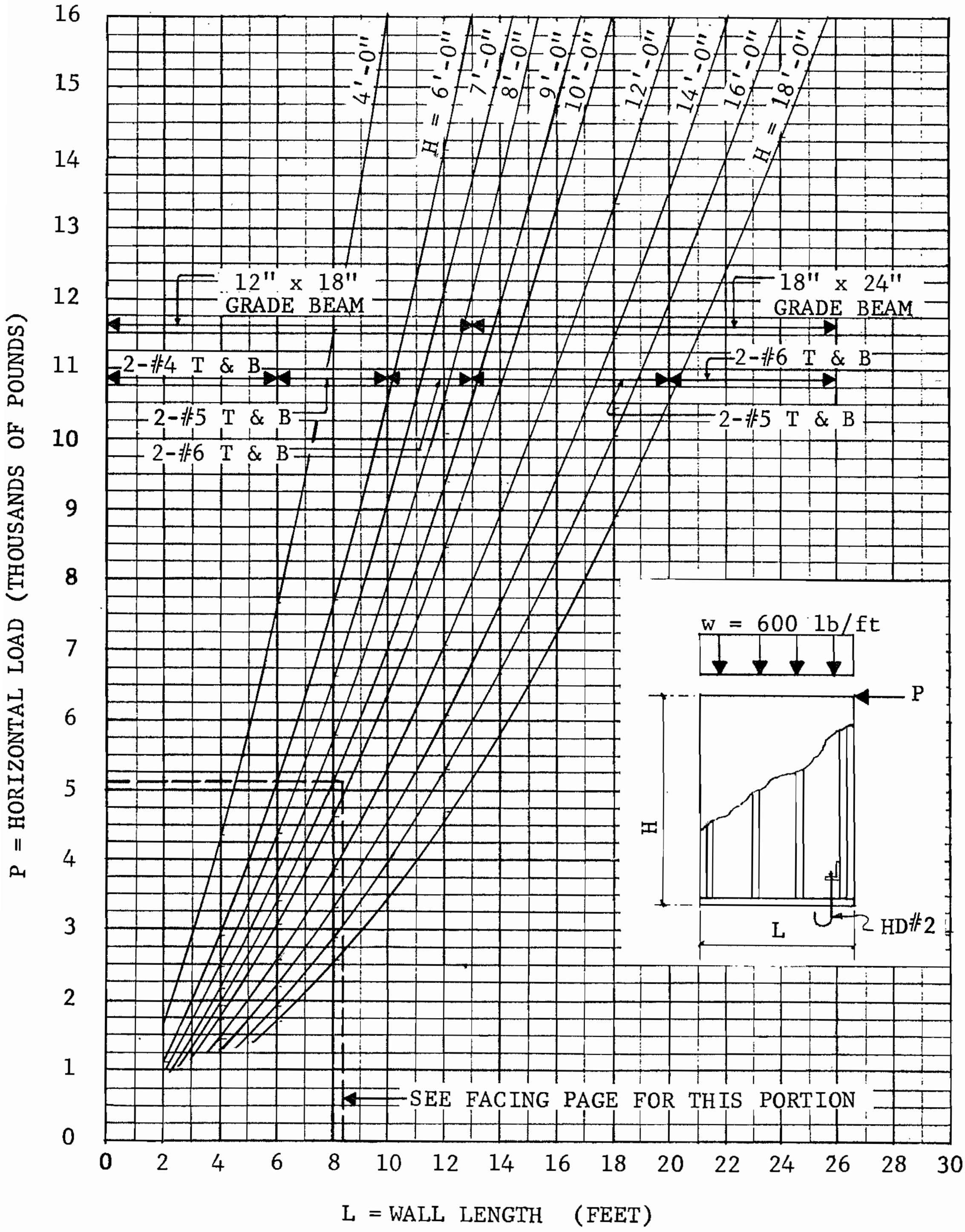
HOLD-DOWN GRAPH 2.F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 500 lb/ft



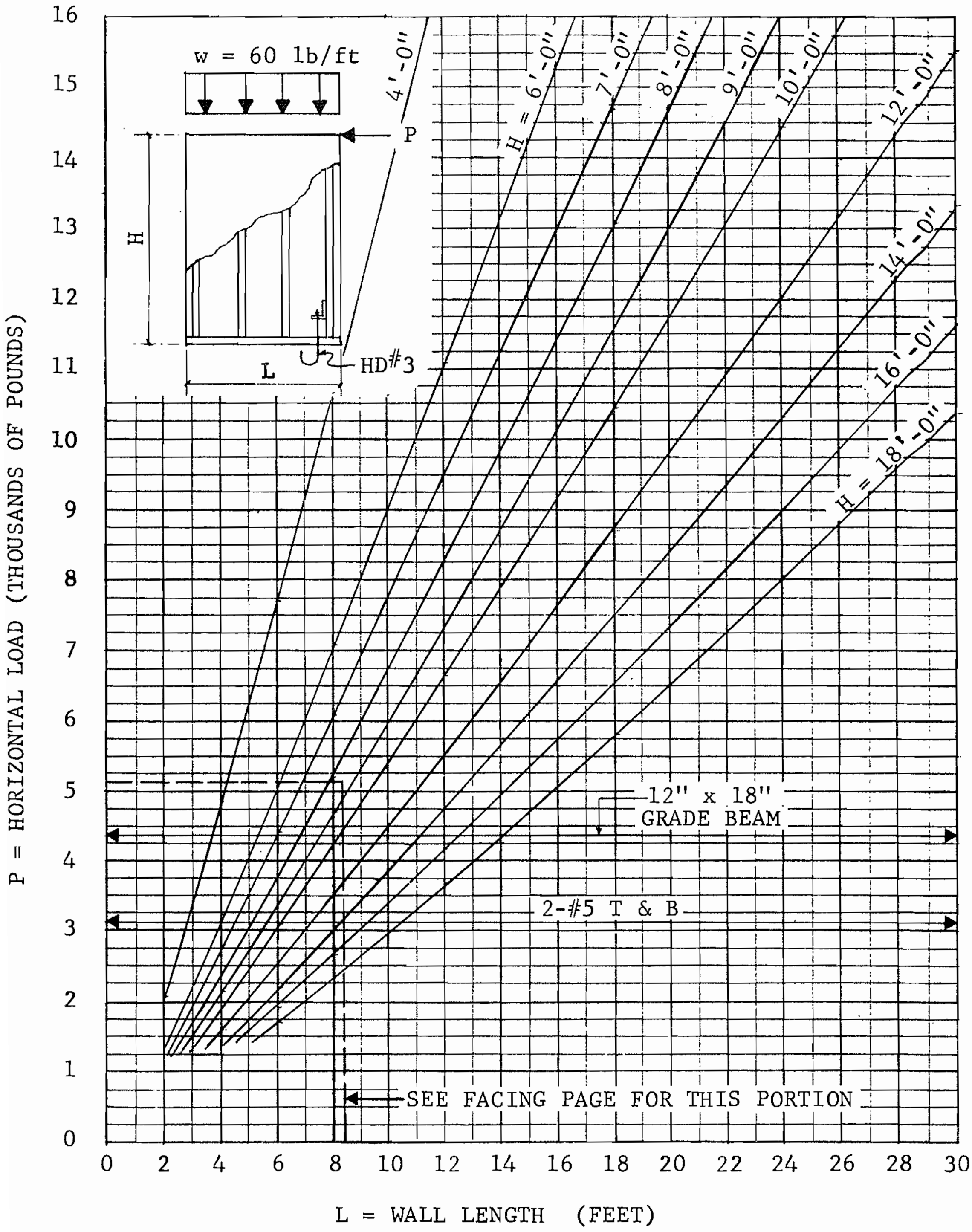
HOLD-DOWN GRAPH 2.G

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 600 lb/ft



HOLD-DOWN GRAPH 2.G

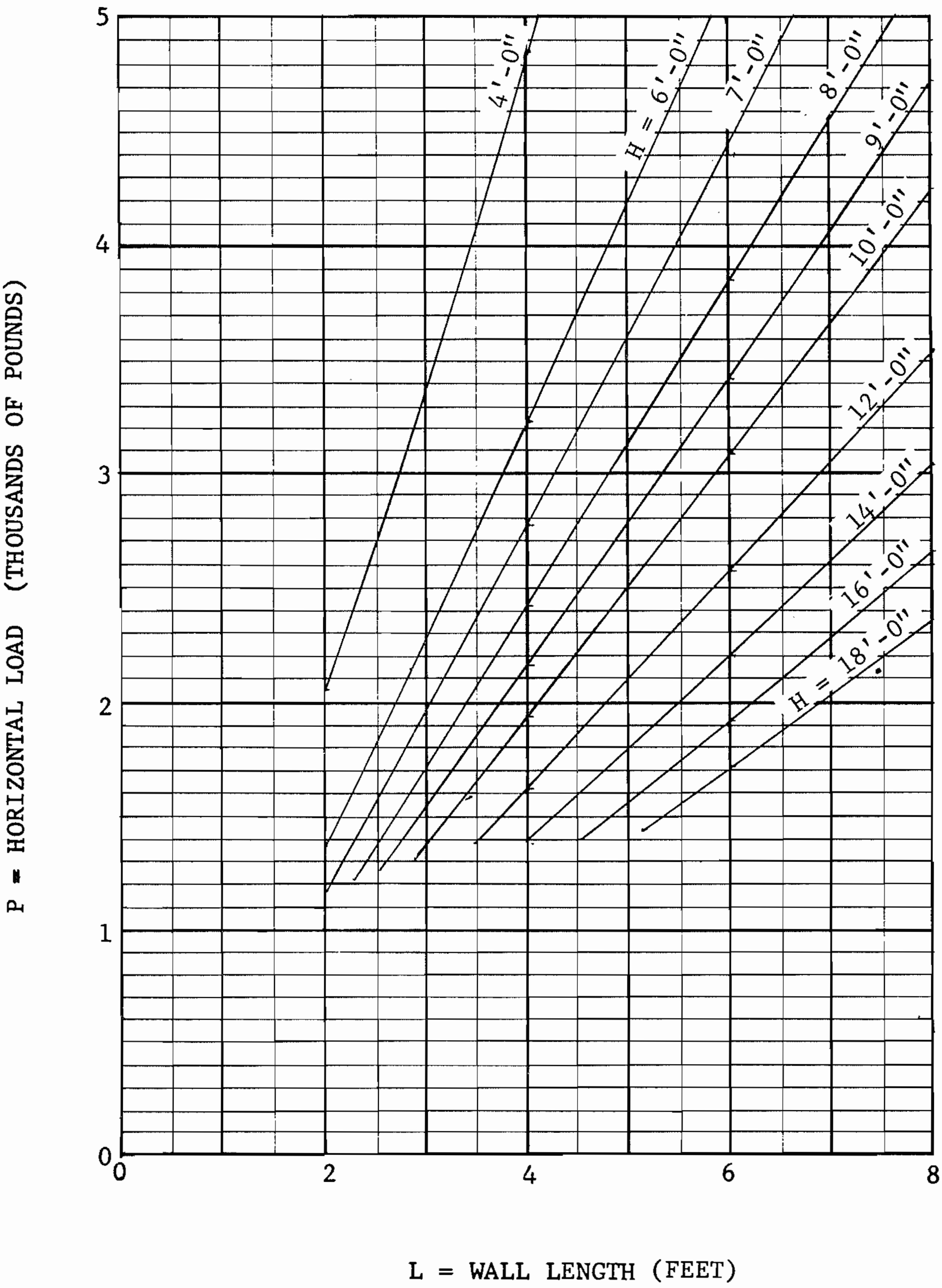
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 2 – w = 600 lb/ft



HOLD-DOWN GRAPH 3.A

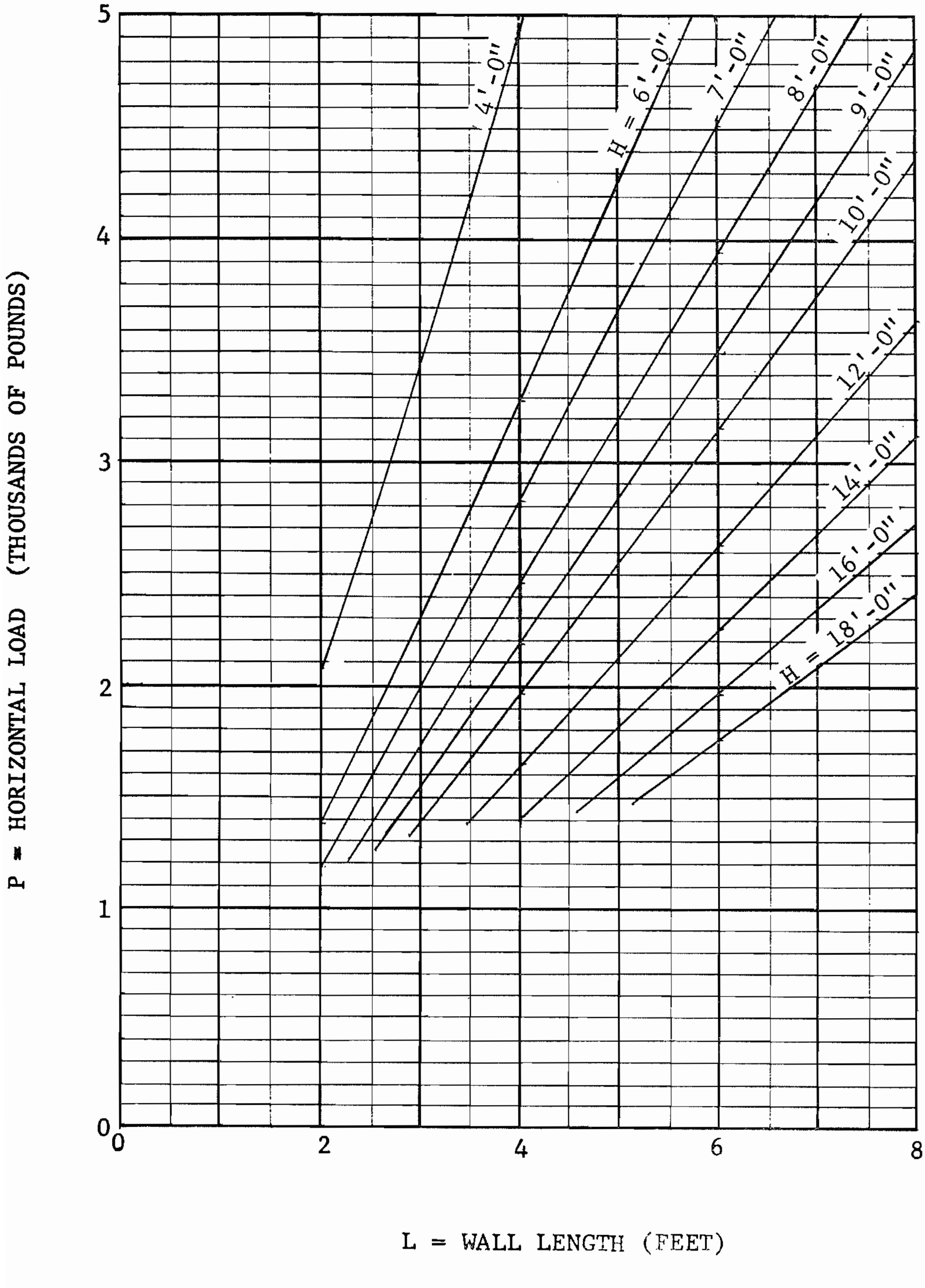
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 3 – w = 60 lb/ft



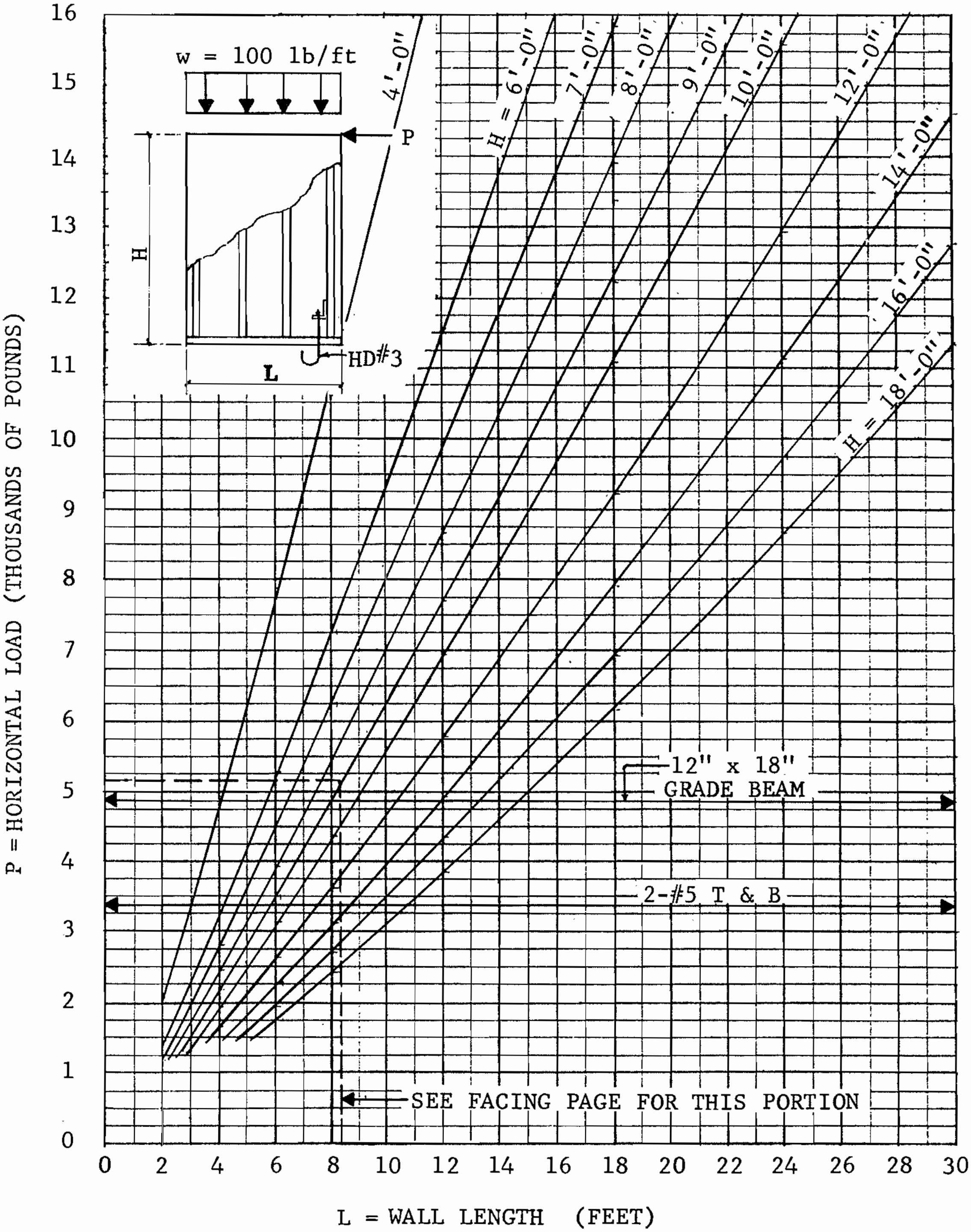
HOLD-DOWN GRAPH 3.A

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 60 lb/ft



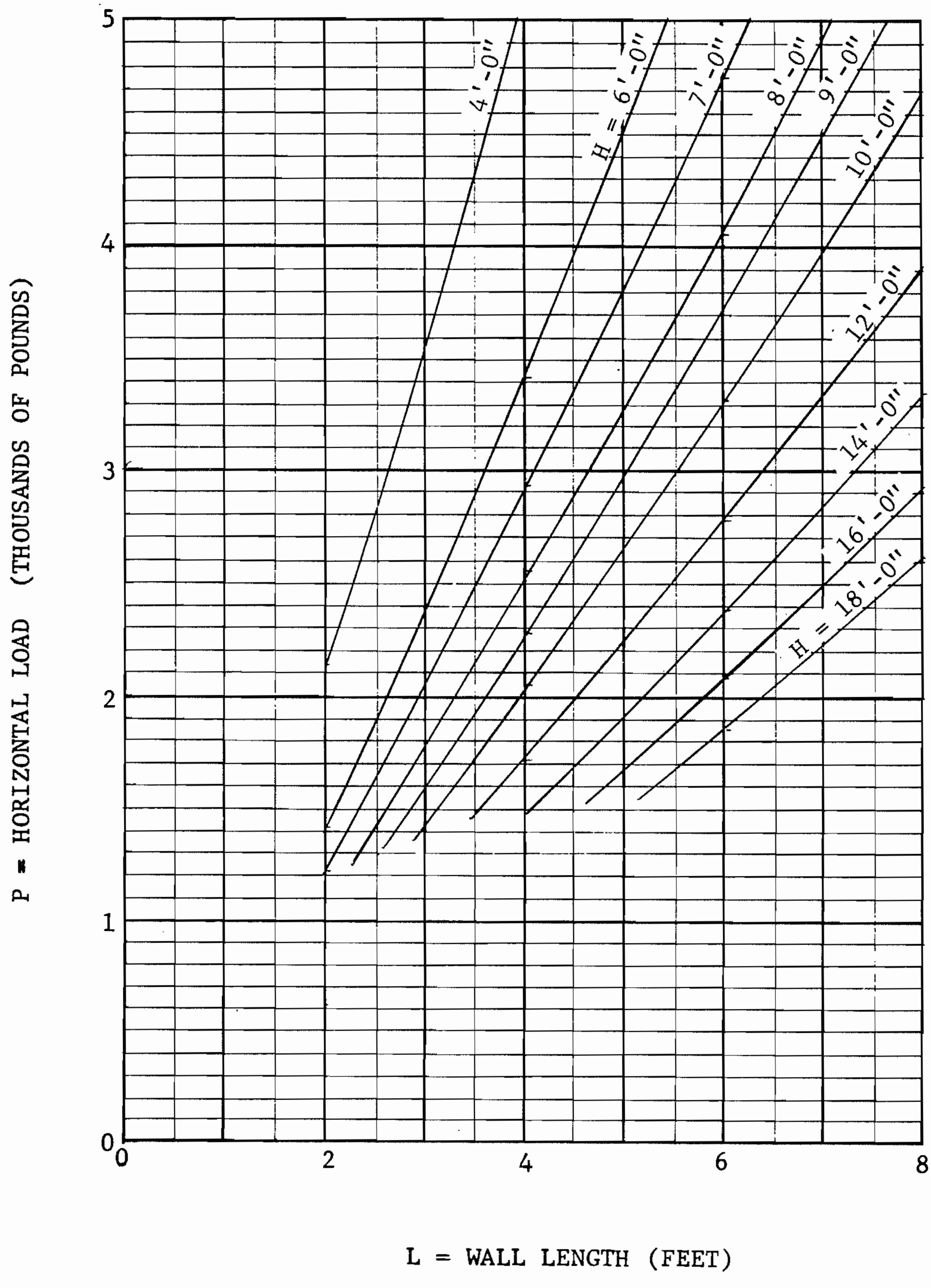
HOLD-DOWN GRAPH 3.B

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 — w = 100 lb/ft



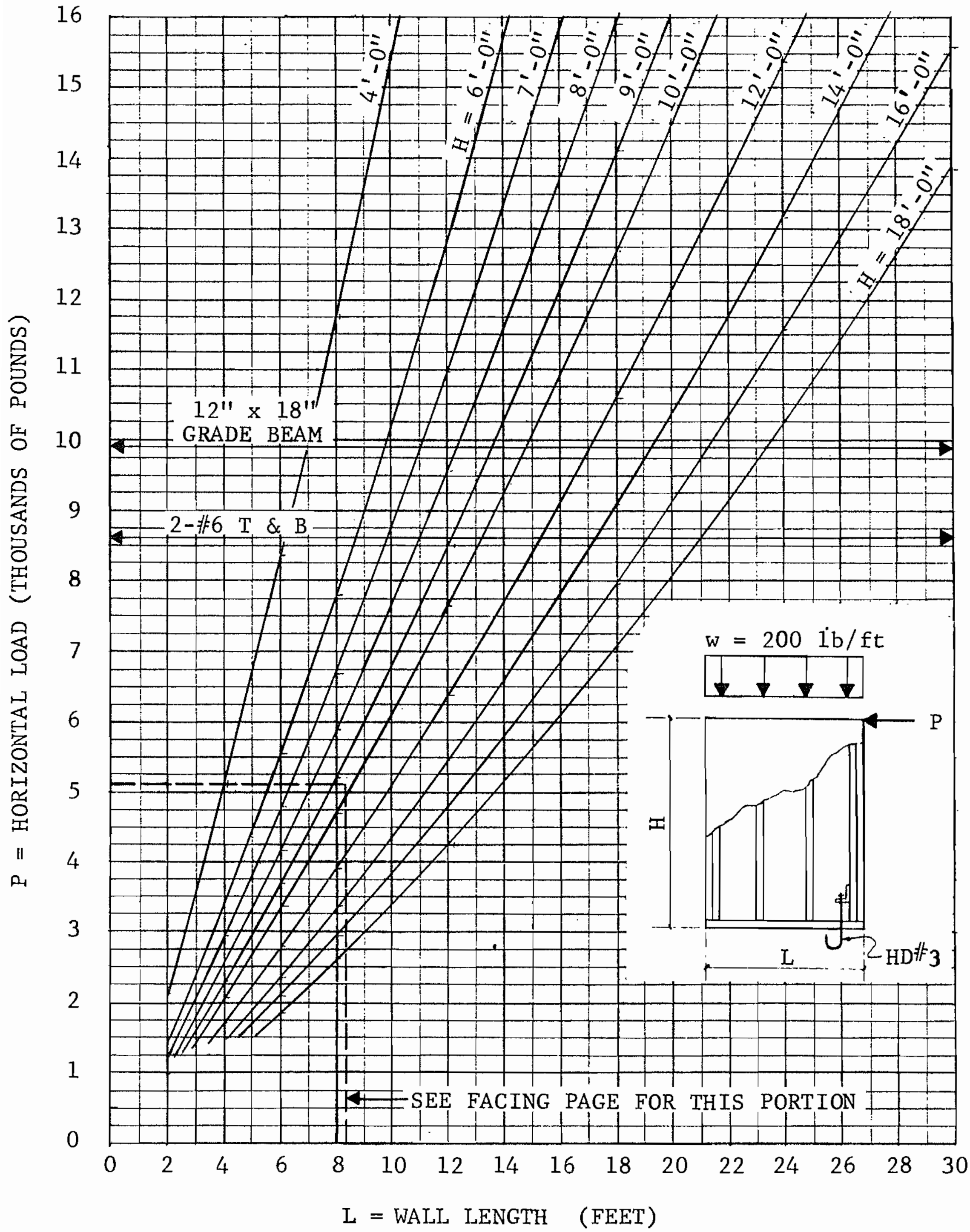
HOLD-DOWN GRAPH 3.B

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 100 lb/ft



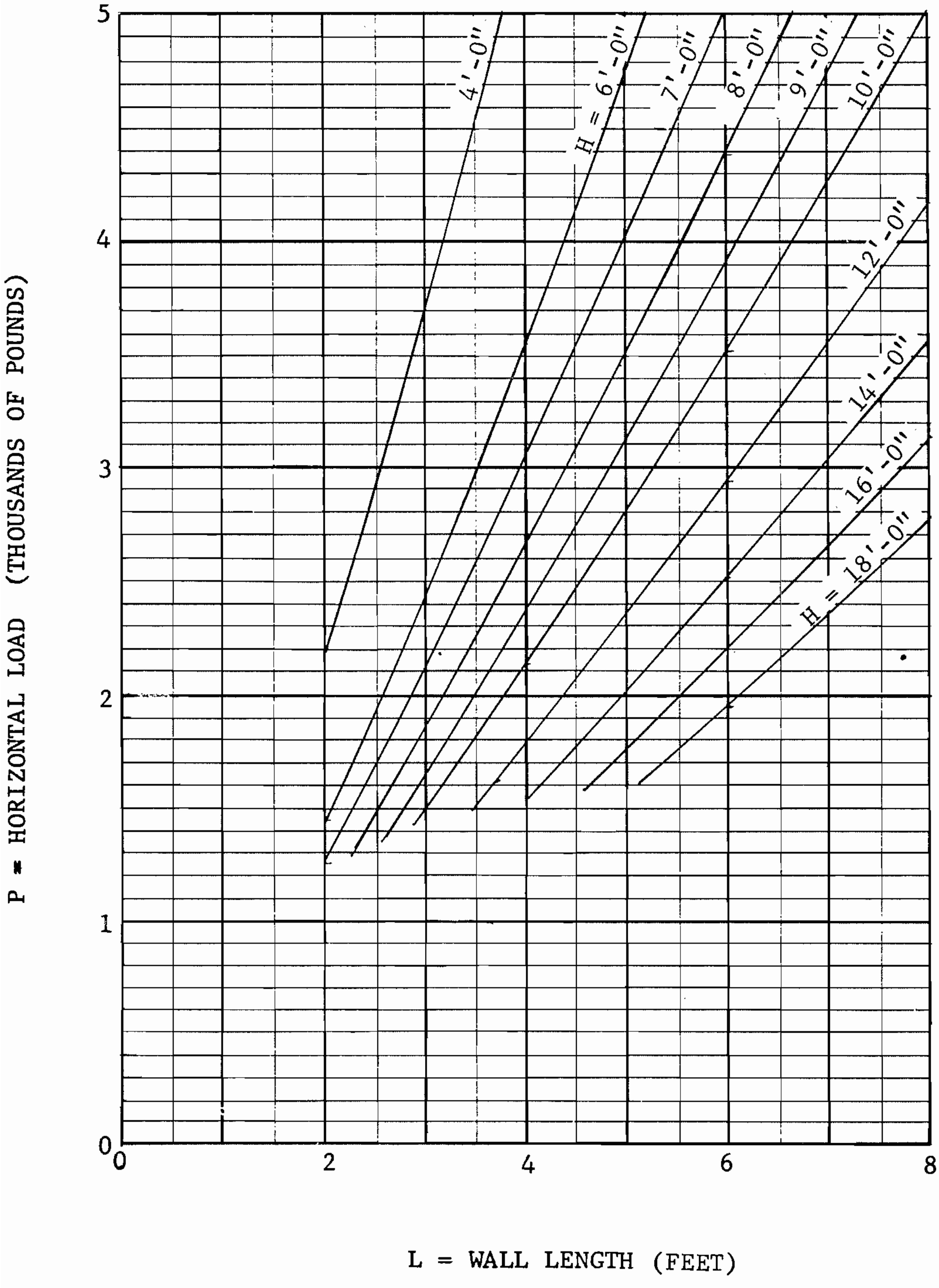
HOLD-DOWN GRAPH 3.C

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 200 lb/ft



HOLD-DOWN GRAPH 3.C

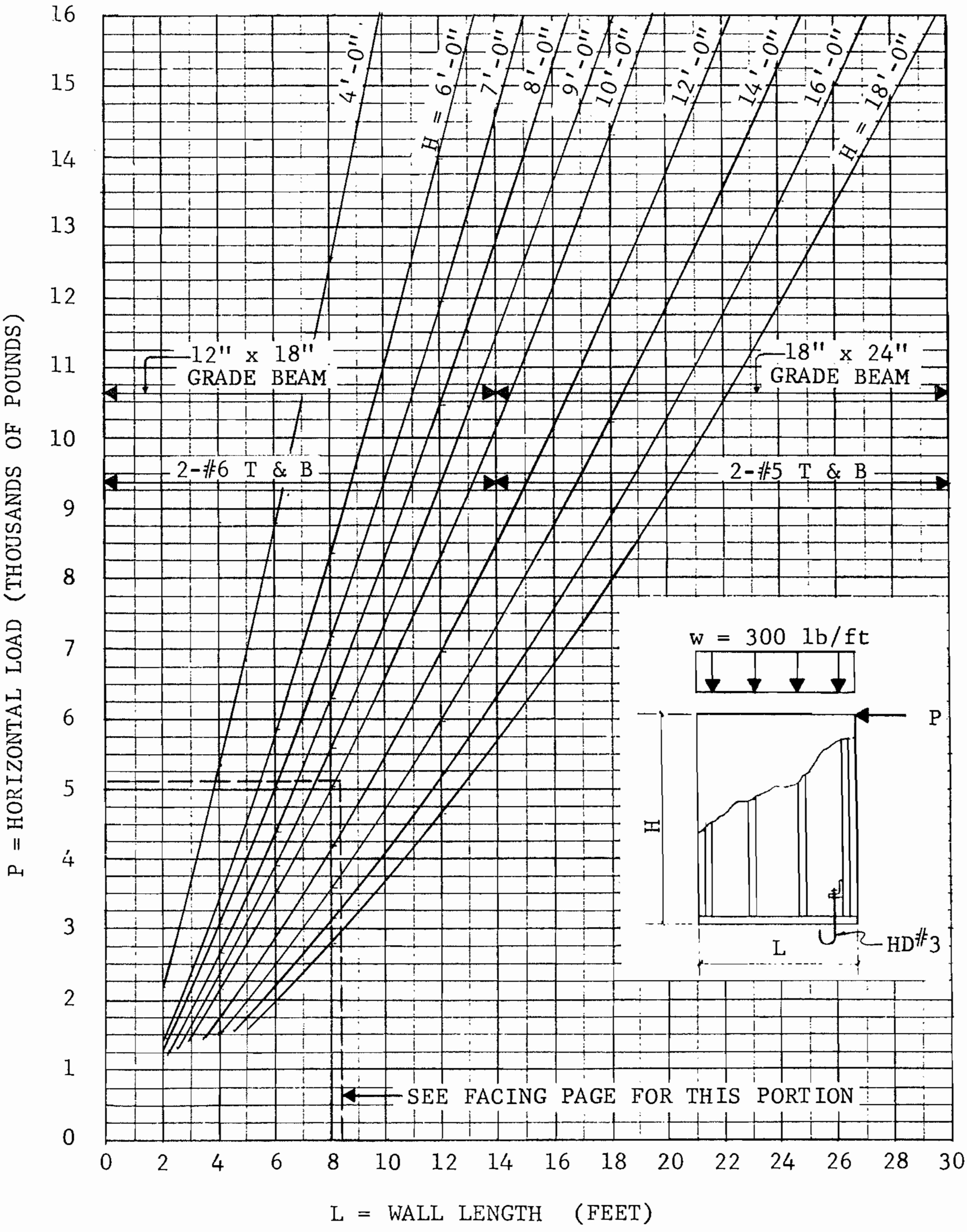
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 200 lb/ft



HOLD-DOWN GRAPH 3.D

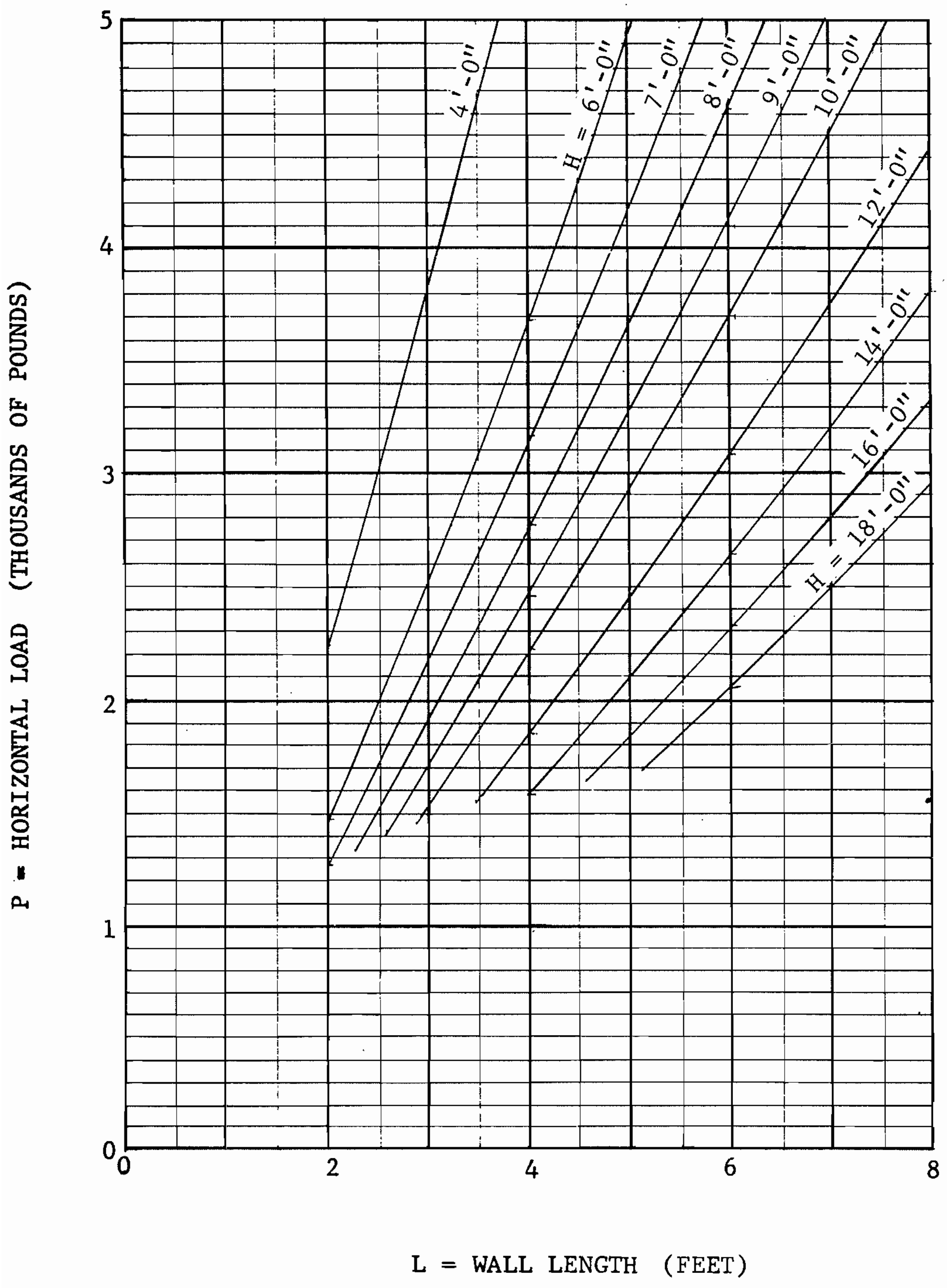
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 3 – w = 300 lb/ft



HOLD-DOWN GRAPH 3.D

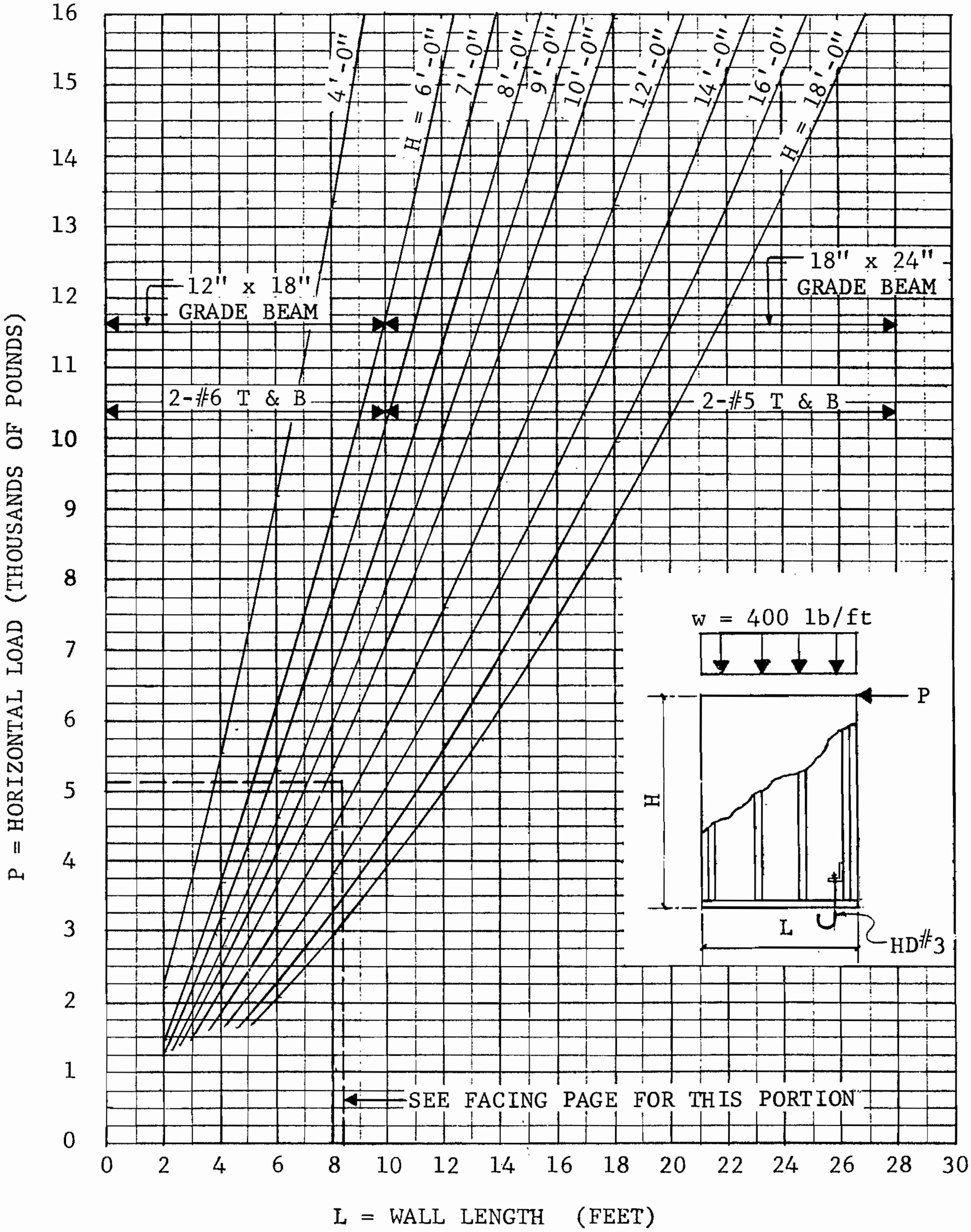
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 — w = 300 lb/ft



HOLD-DOWN GRAPH 3.E

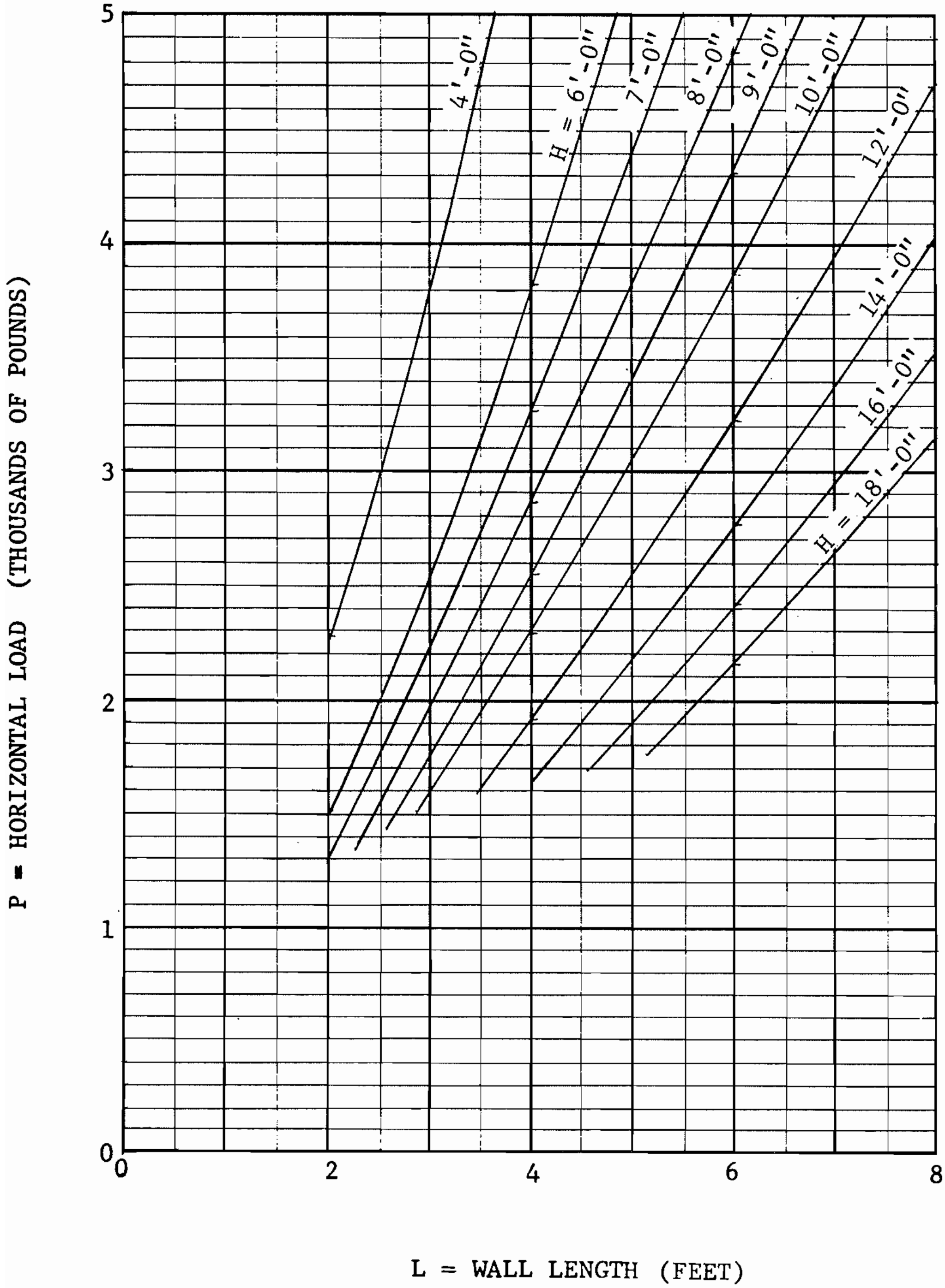
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 3 – w = 400 lb/ft



HOLD-DOWN GRAPH 3.E

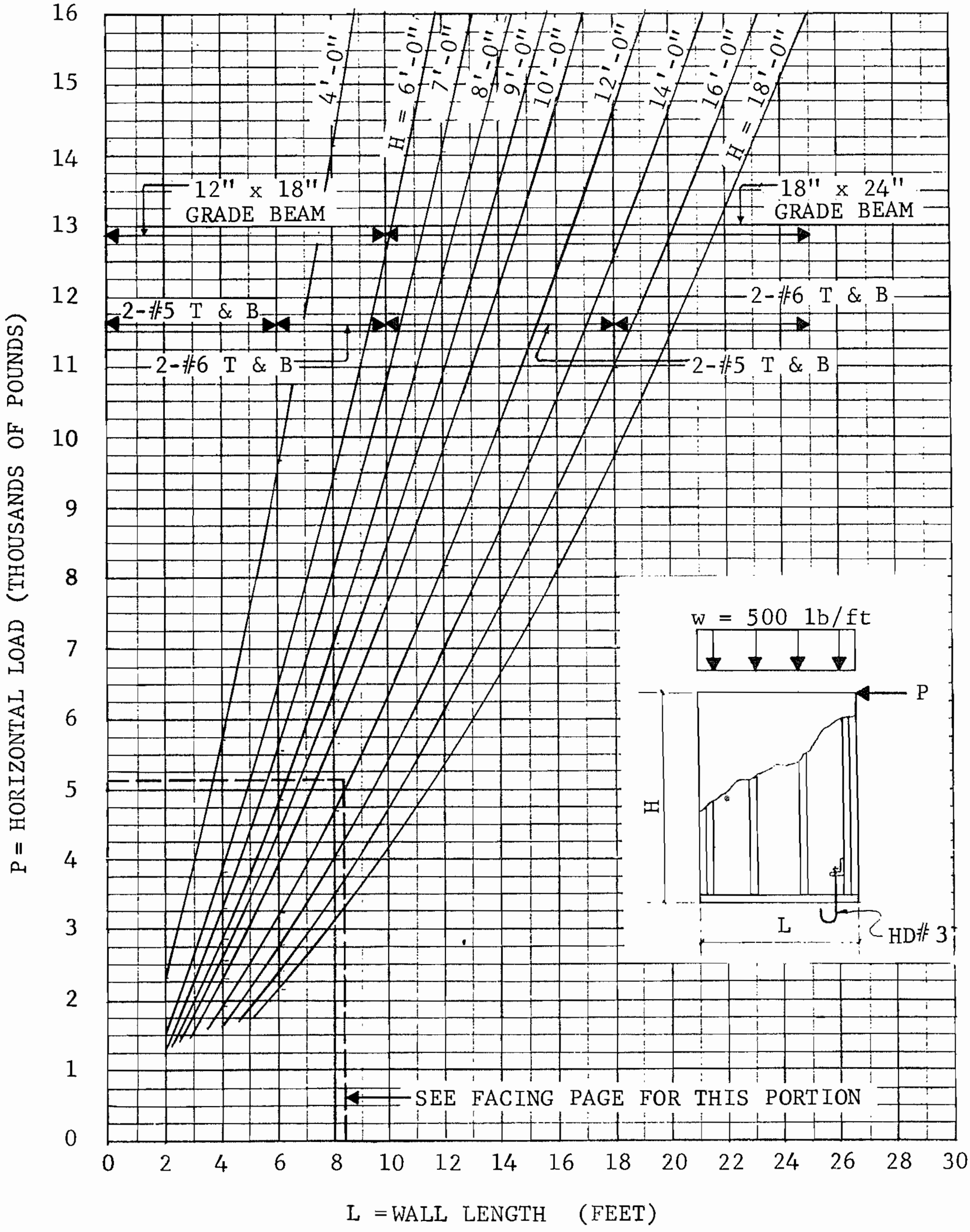
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 400 lb/ft



HOLD-DOWN GRAPH 3.F

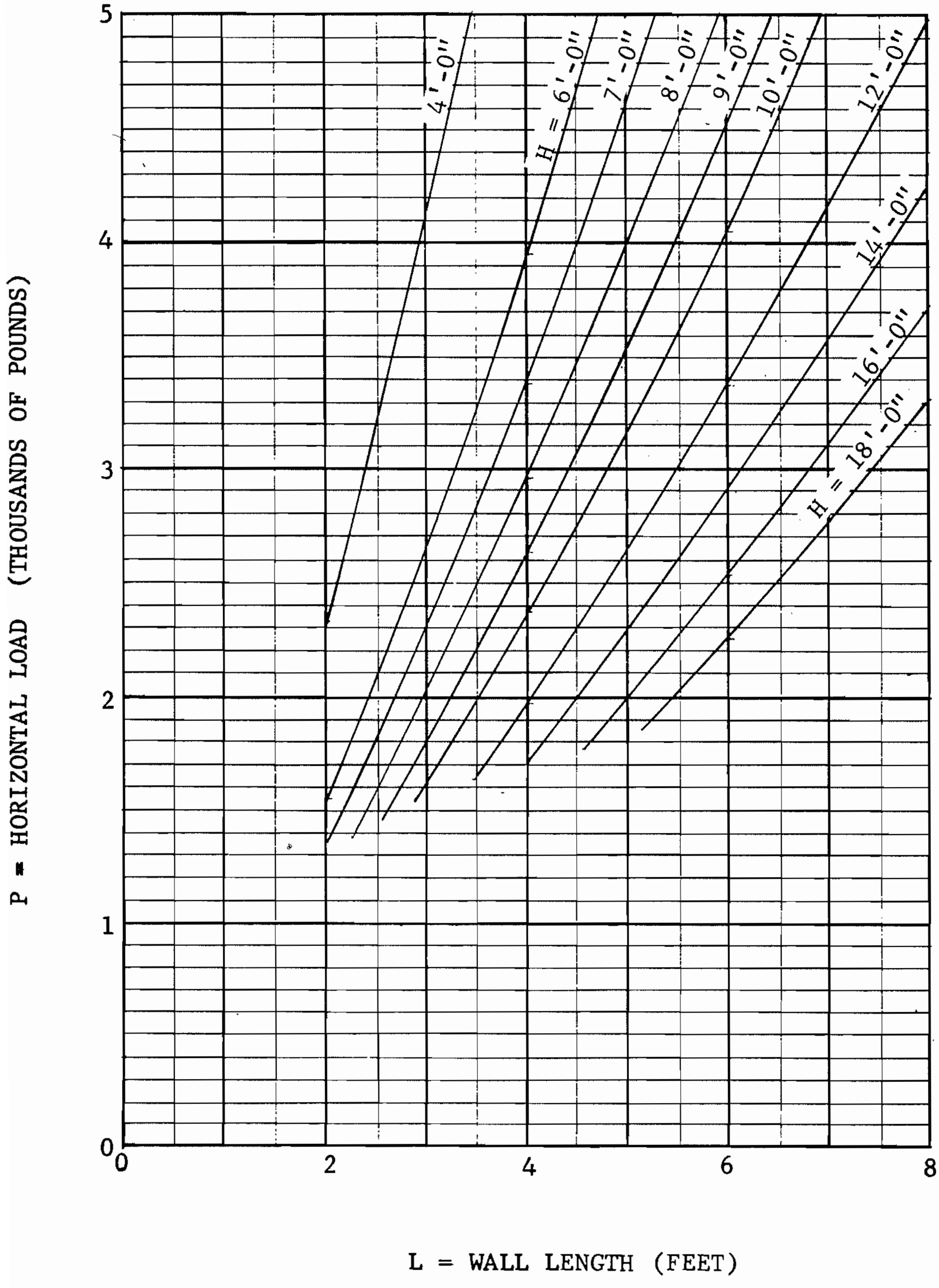
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 3 – w = 500 lb/ft



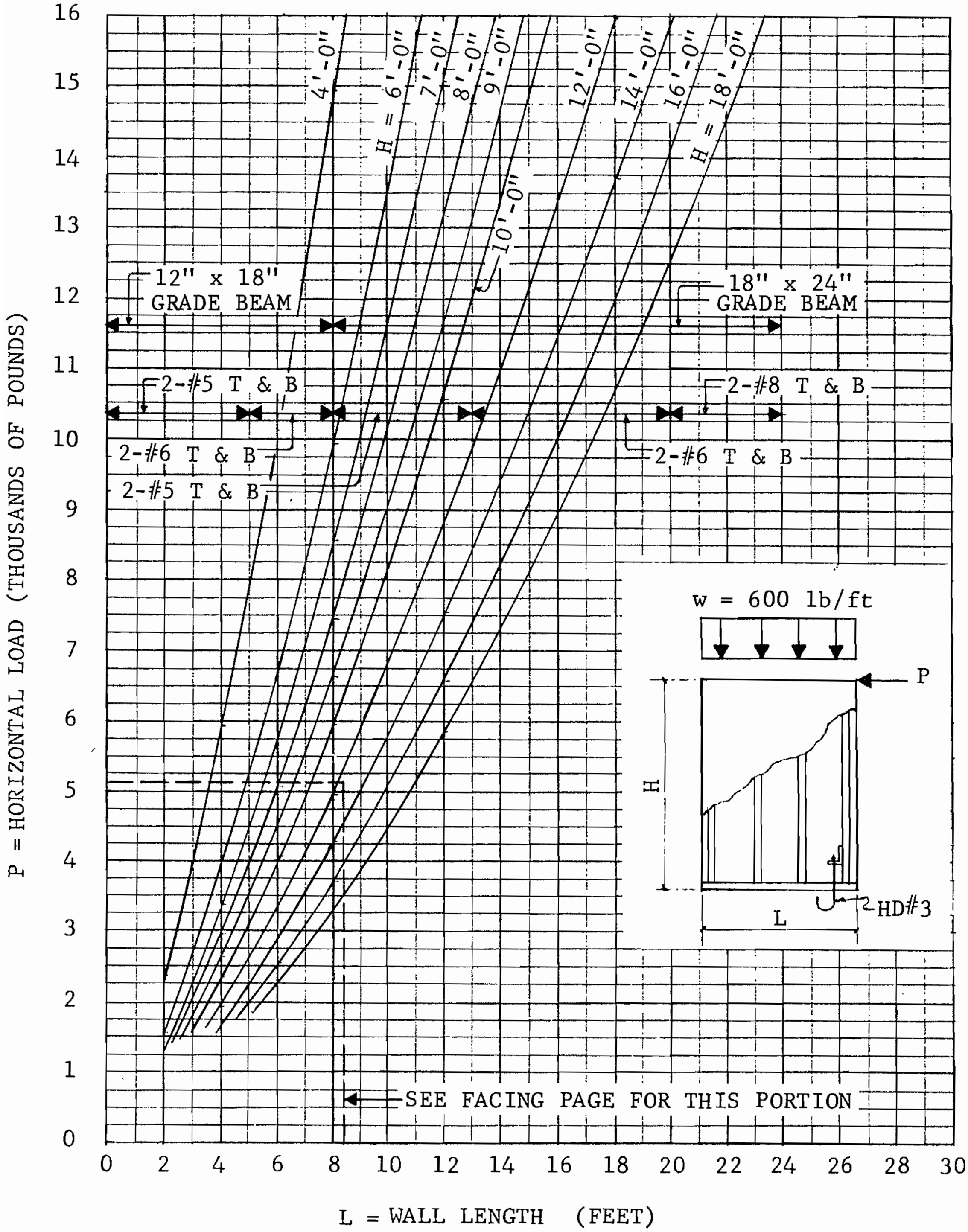
HOLD-DOWN GRAPH 3.F

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 500 lb/ft



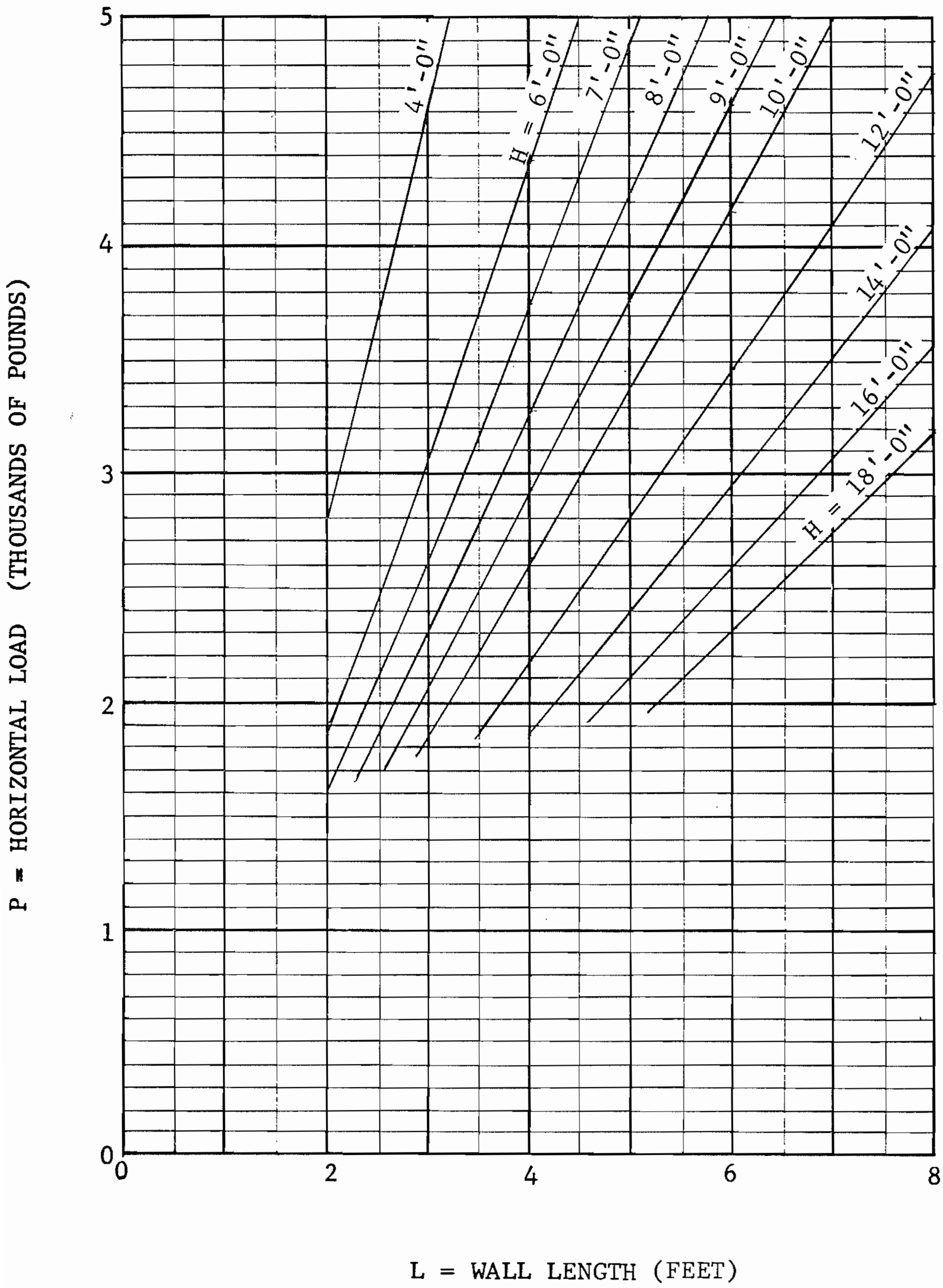
HOLD-DOWN GRAPH 3.G

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 600 lb/ft



HOLD-DOWN GRAPH 3.G

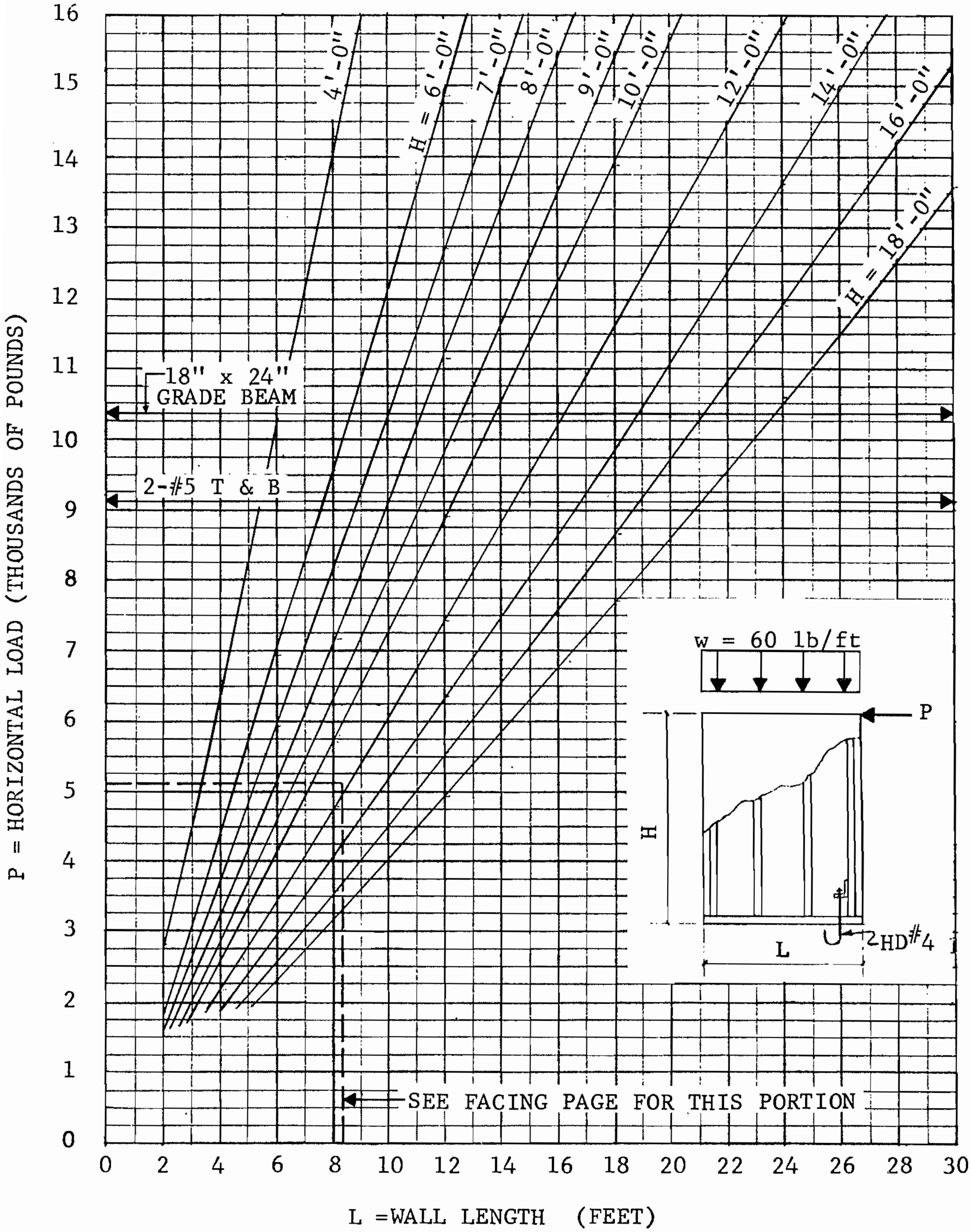
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 3 – w = 600 lb/ft



<u>HOLD-DOWN GRAPH 4.A</u>

ALLOWABLE HORIZONTAL LOAD (P) FOR:

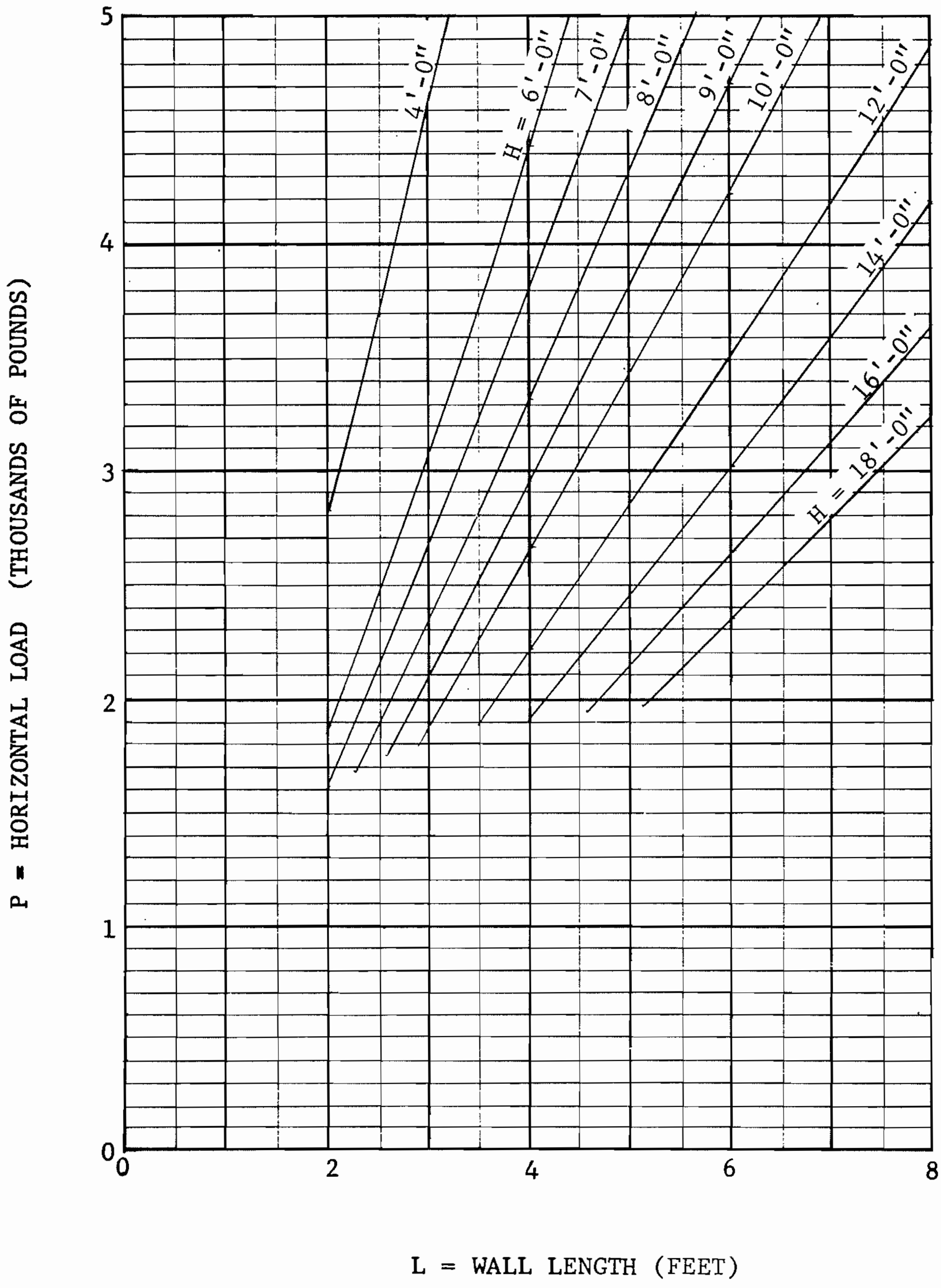
HOLD-DOWN # 4 – w = 60 lb/ft



HOLD-DOWN GRAPH 4.A

ALLOWABLE HORIZONTAL LOAD (P) FOR:

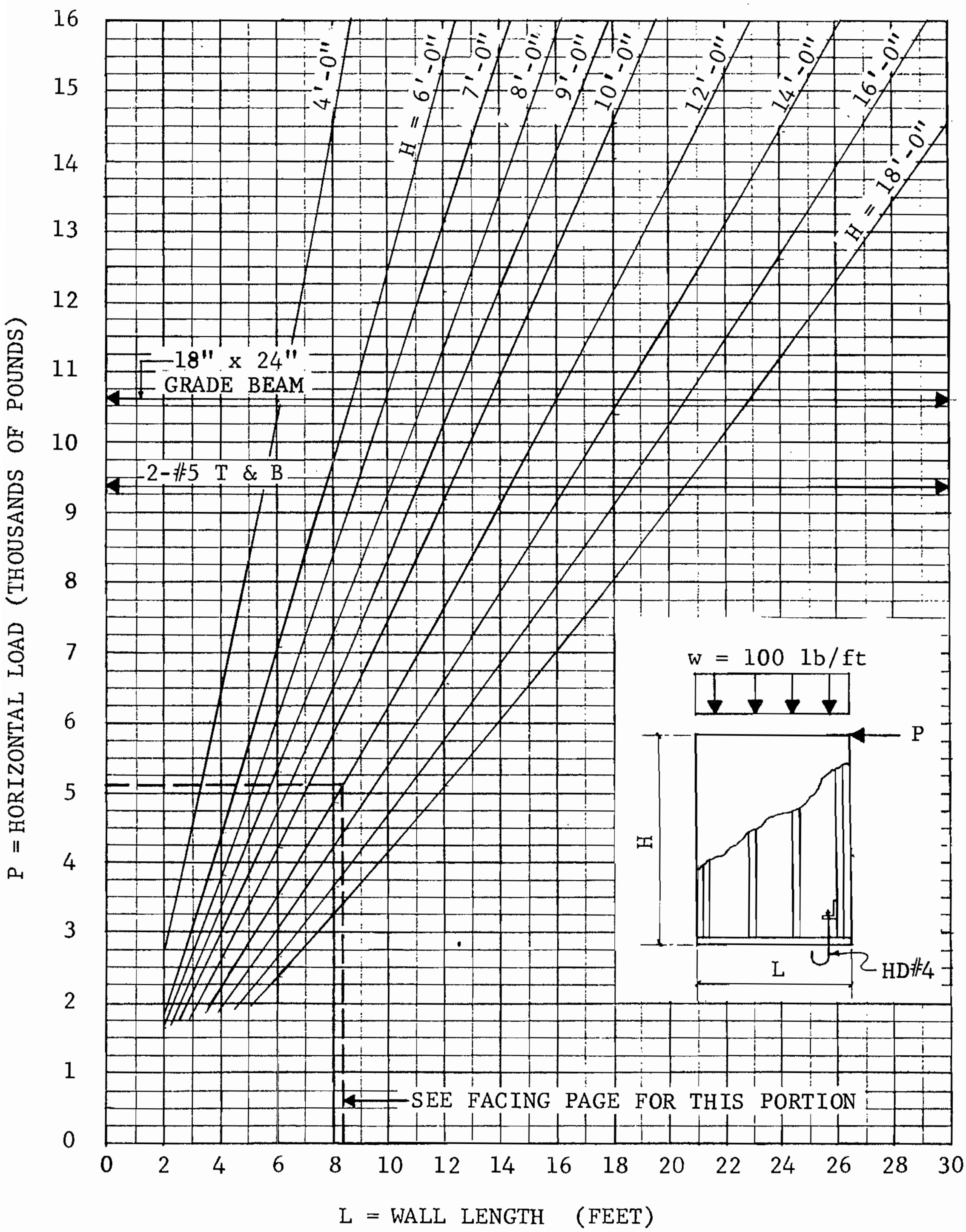
HOLD-DOWN # 4 – w = 60 lb/ft



HOLD-DOWN GRAPH 4.B

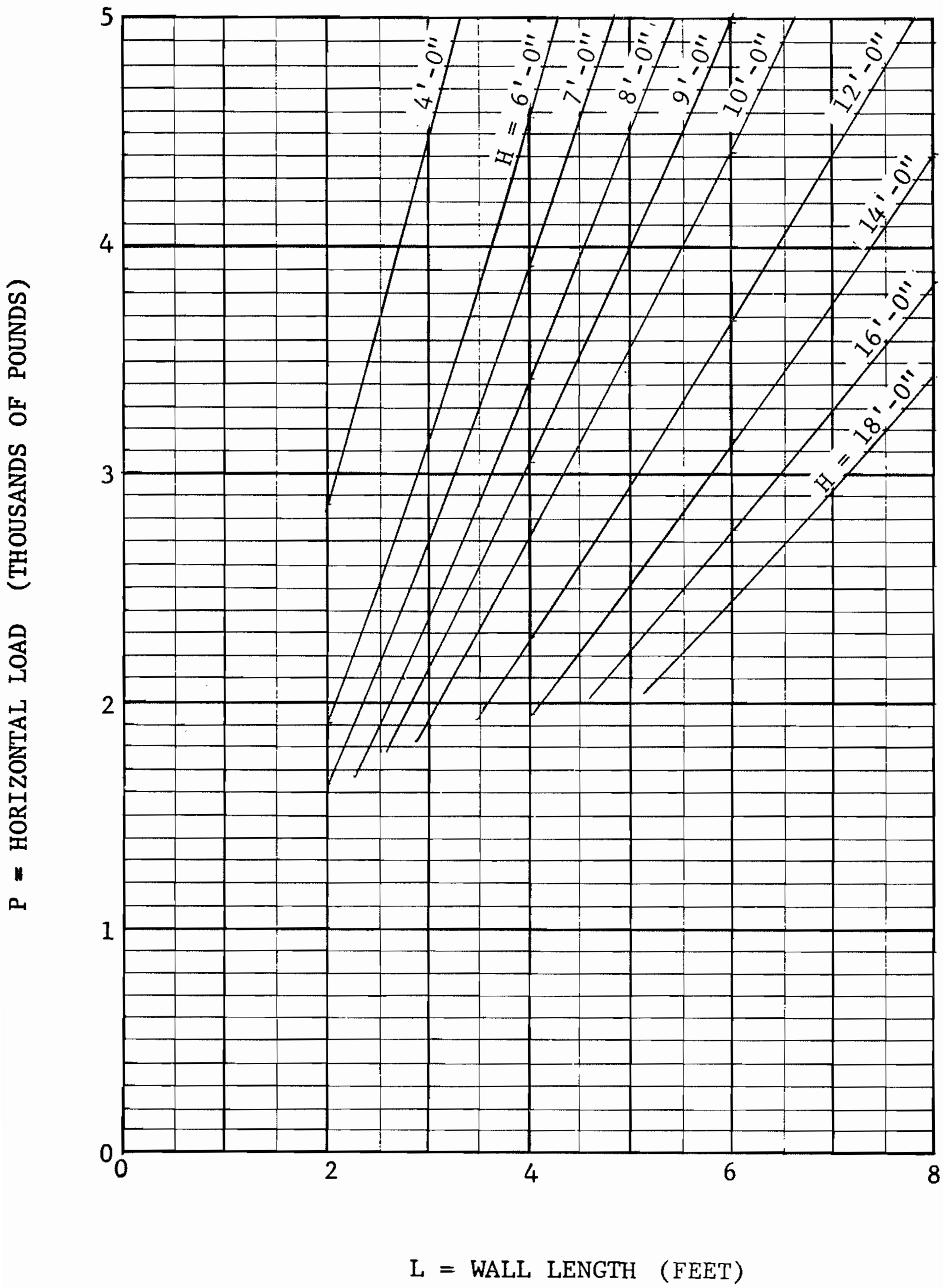
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 4 – w = 100 lb/ft



HOLD-DOWN GRAPH 4.B

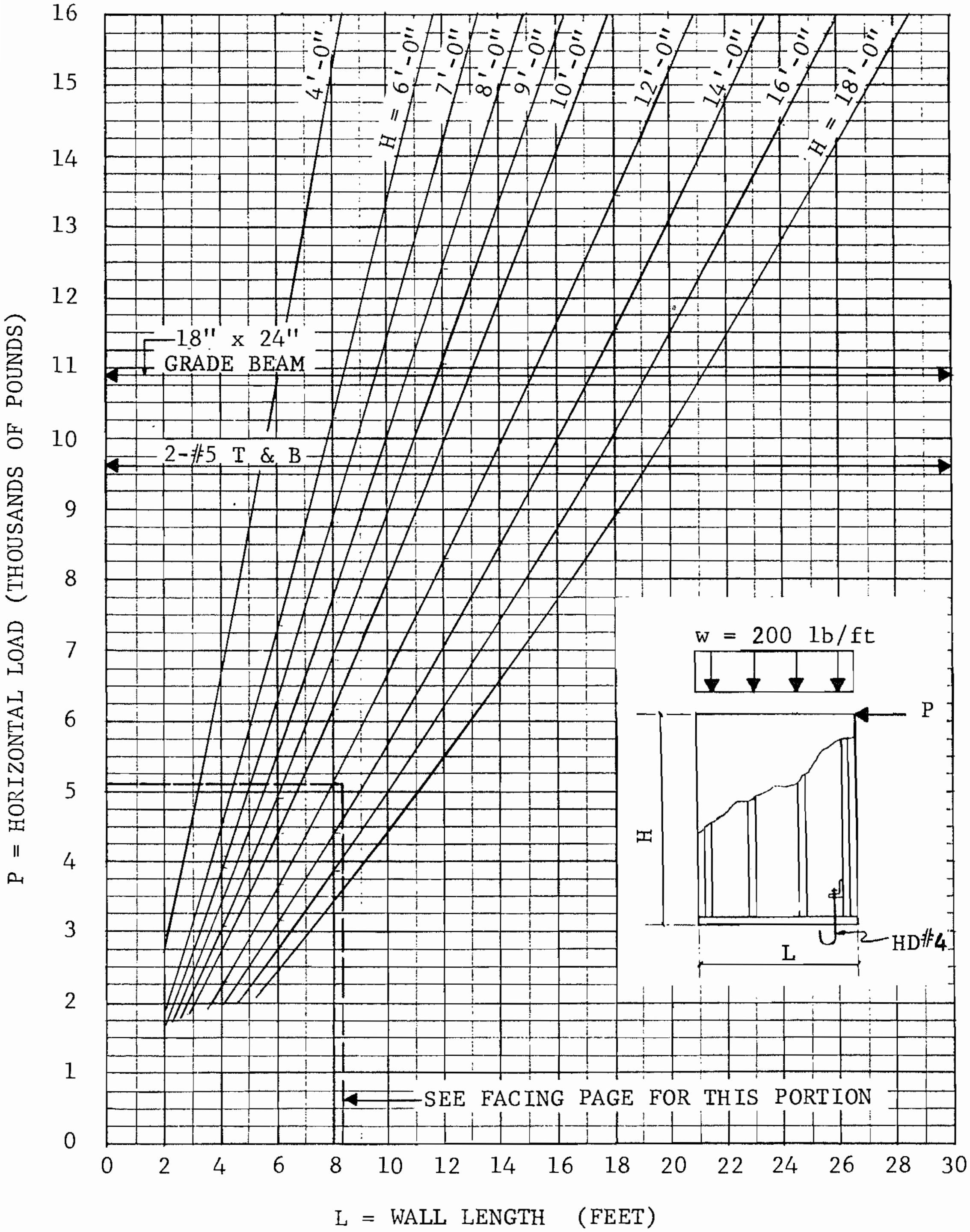
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 100 lb/ft



HOLD-DOWN GRAPH 4.C

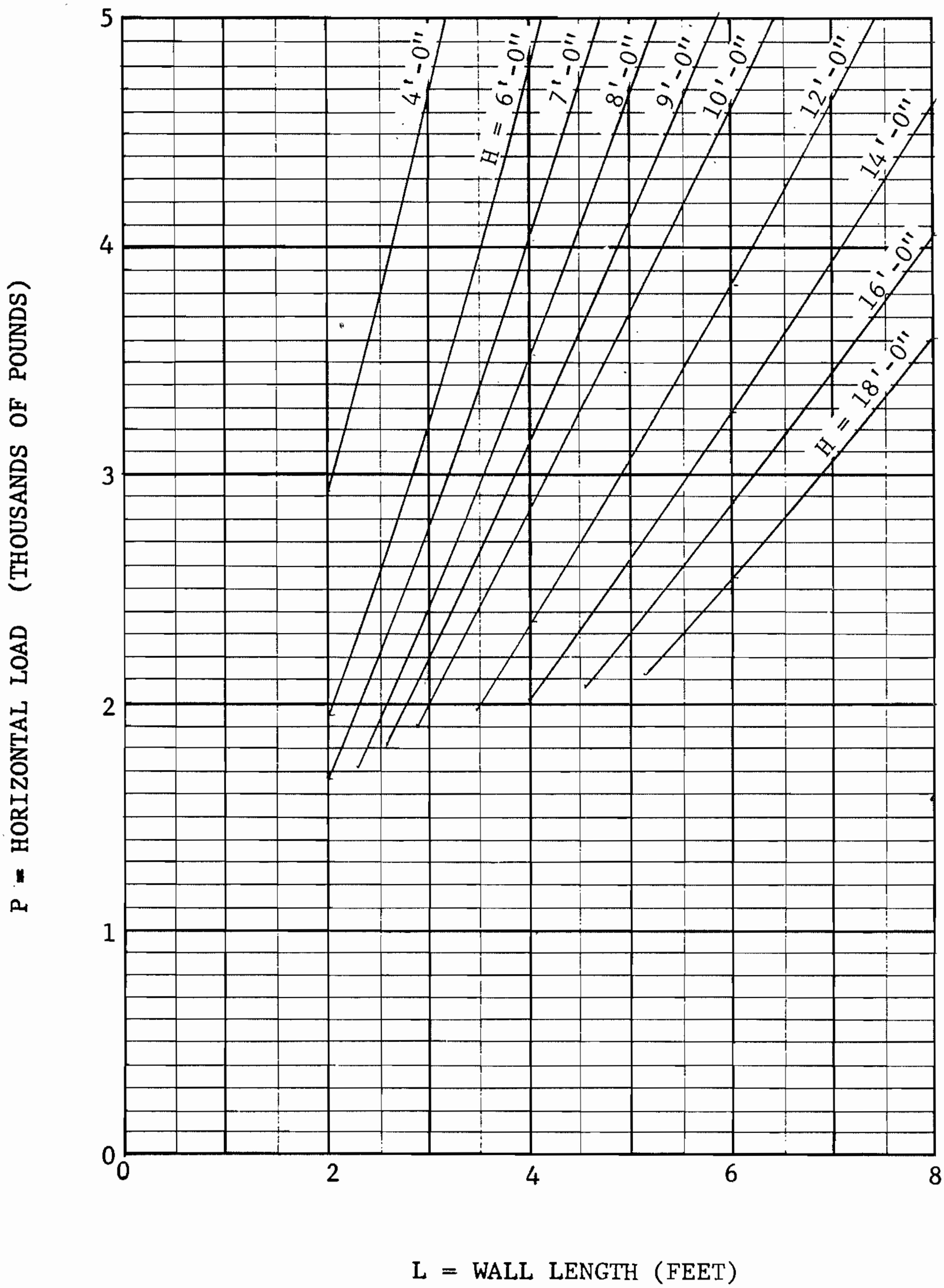
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 4 – w = 200 lb/ft



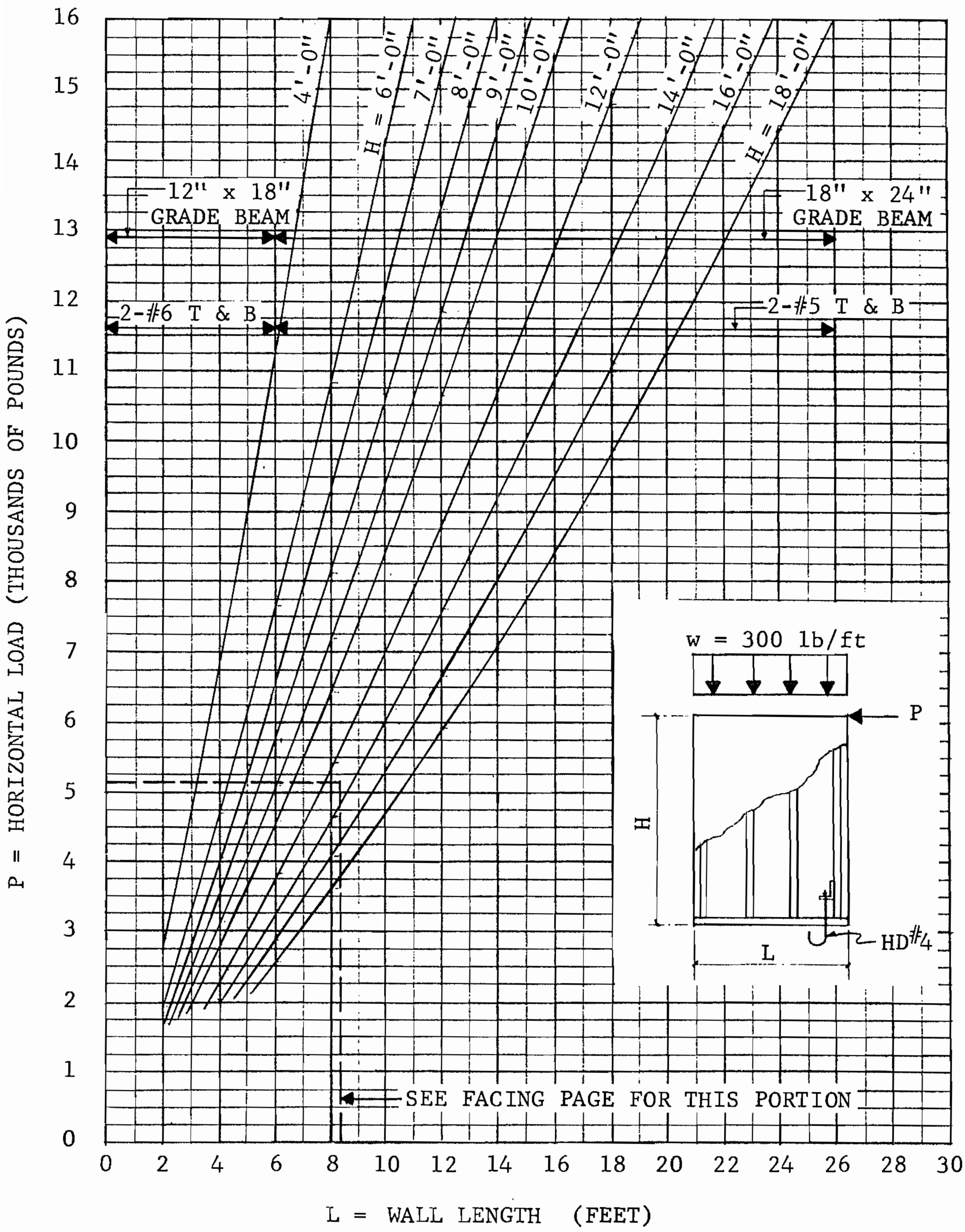
HOLD-DOWN GRAPH 4.C

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 — w = 200 lb/ft



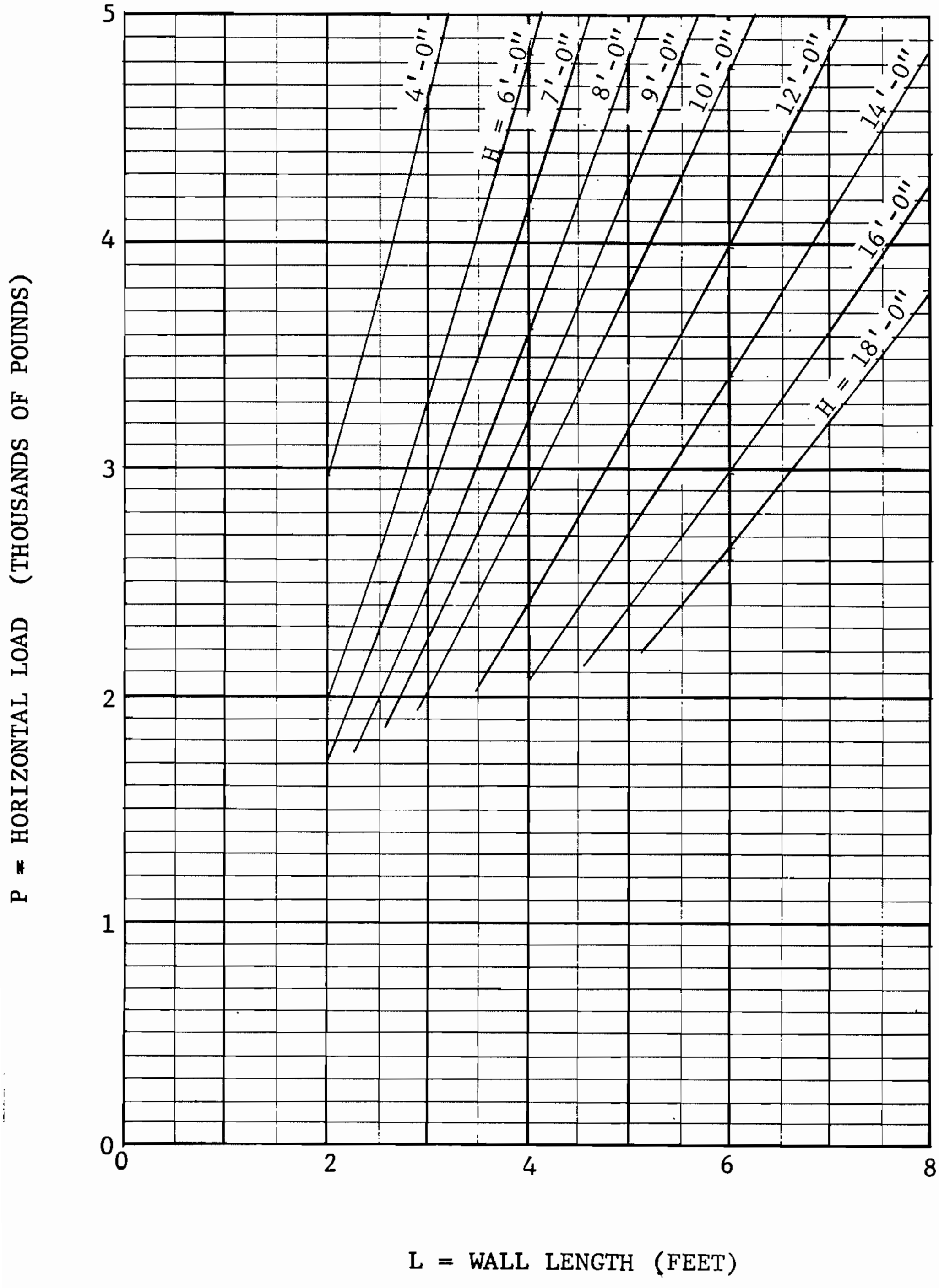
HOLD-DOWN GRAPH 4.D

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 300 lb/ft



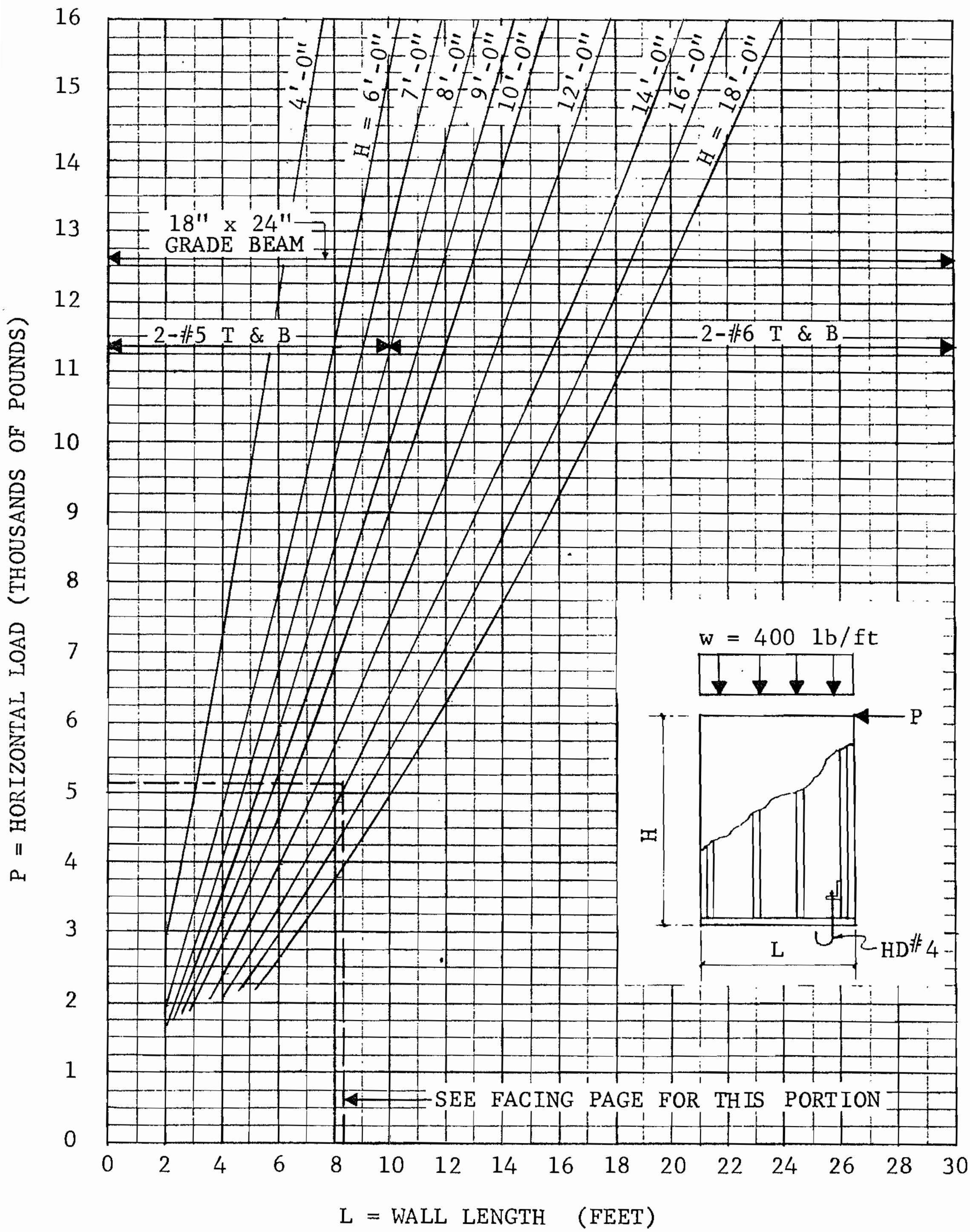
HOLD-DOWN GRAPH 4.D

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 300 lb/ft



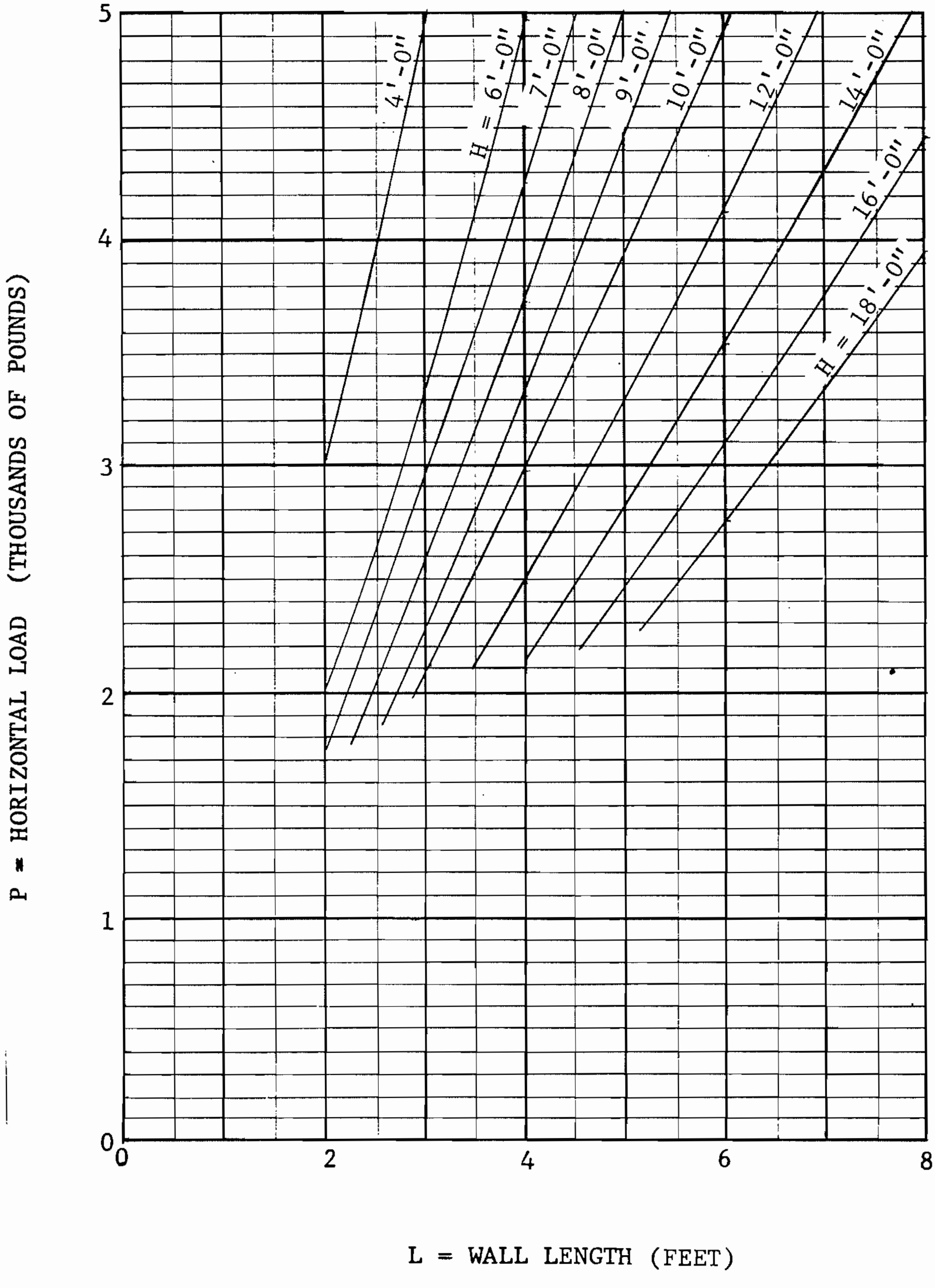
HOLD-DOWN GRAPH 4.E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 400 lb/ft



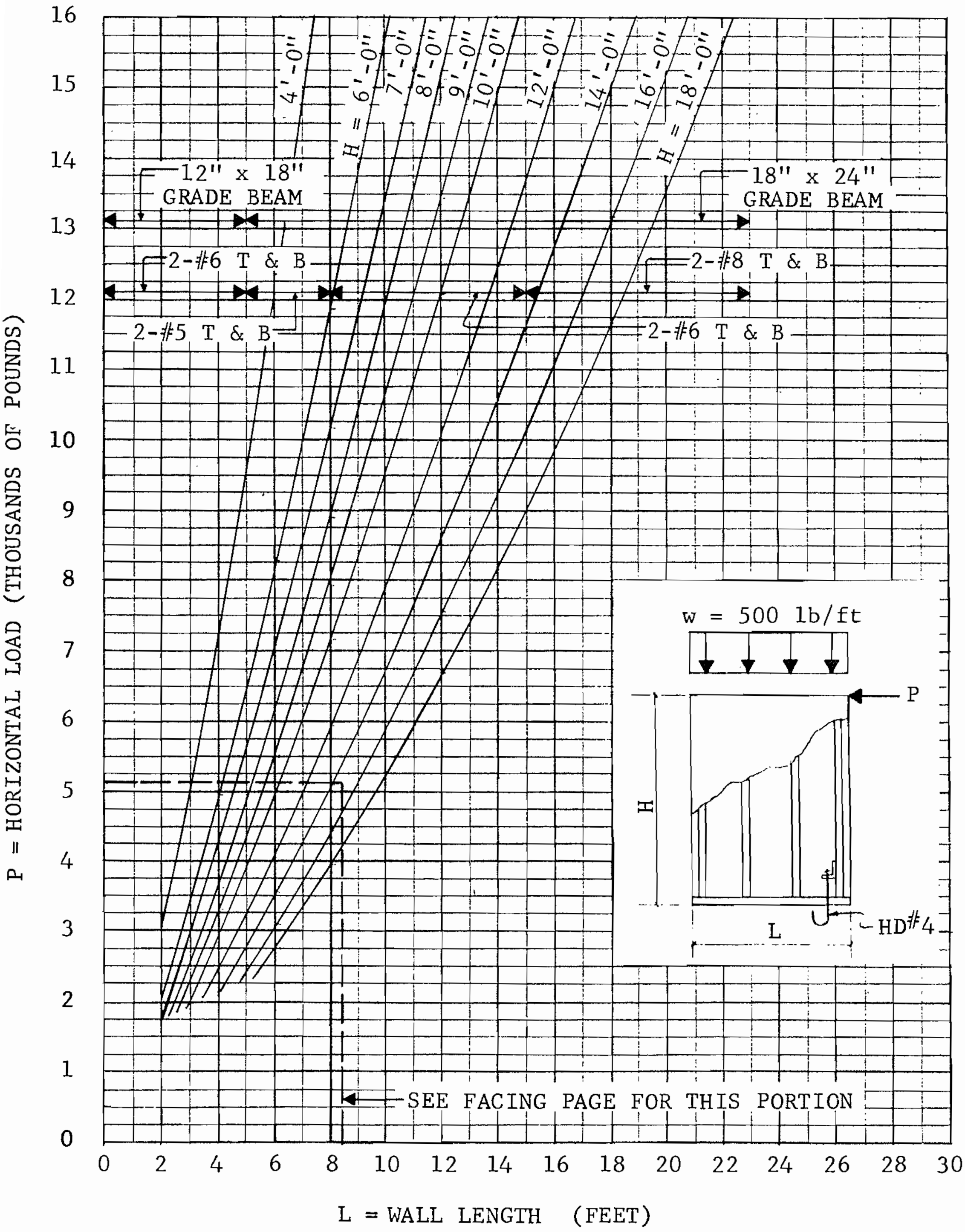
HOLD-DOWN GRAPH 4.E

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 400 lb/ft



HOLD-DOWN GRAPH 4.F

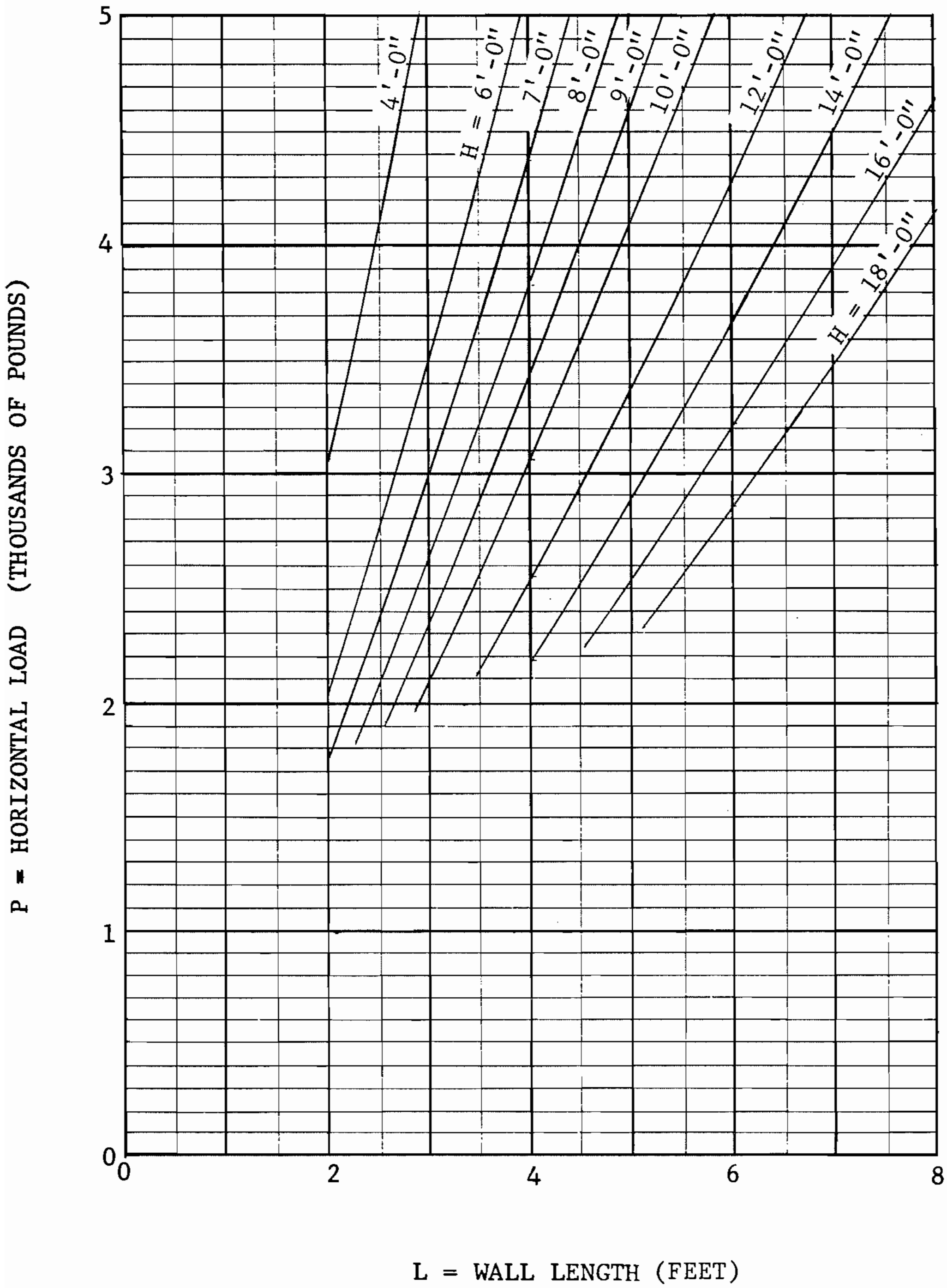
ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 500 lb/ft



HOLD-DOWN GRAPH 4.F

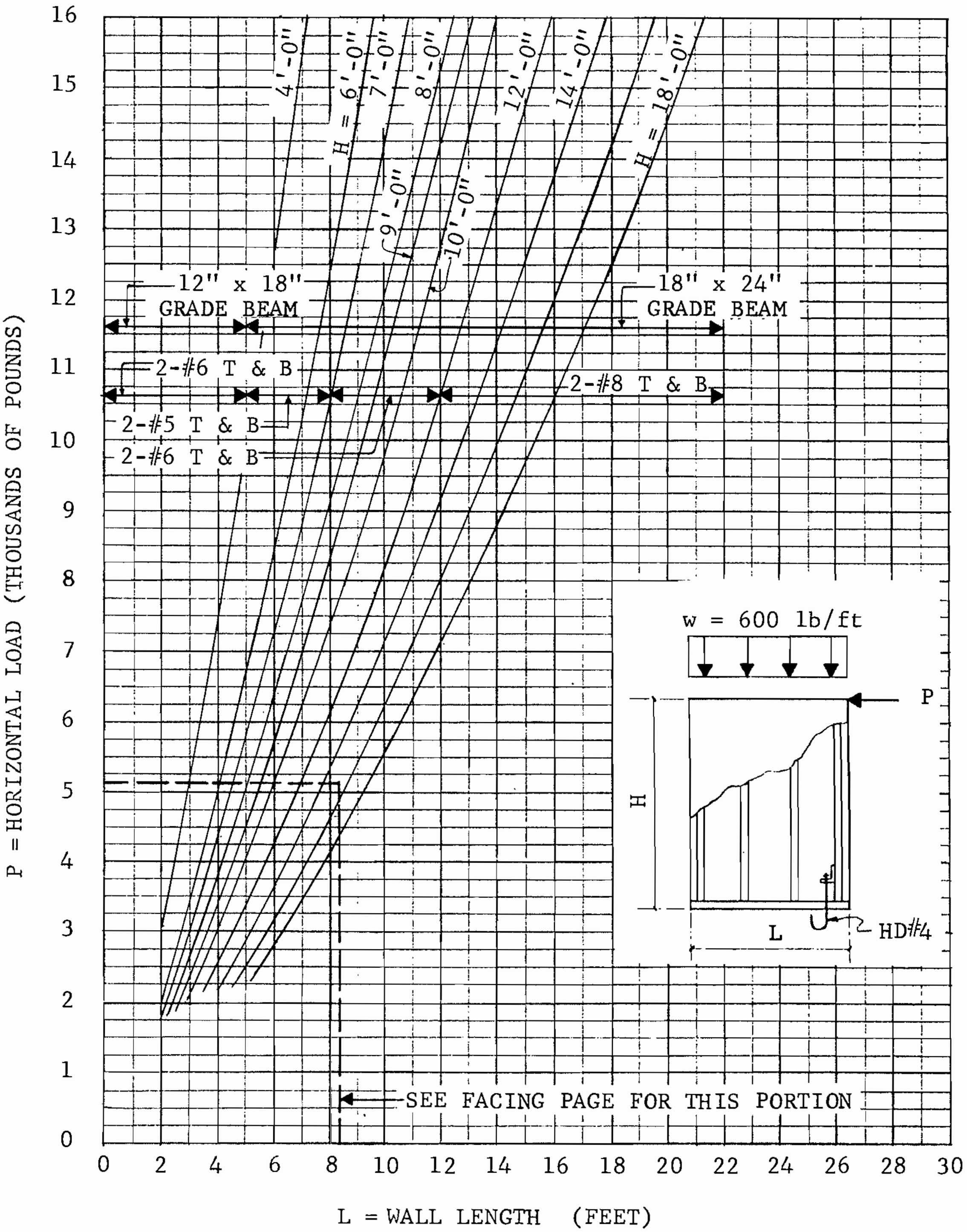
ALLOWABLE HORIZONTAL LOAD (P) FOR:

HOLD-DOWN # 4 – w = 500 lb/ft



HOLD-DOWN GRAPH 4.G

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 — w = 600 lb/ft



HOLD-DOWN GRAPH 4.G

ALLOWABLE HORIZONTAL LOAD (P) FOR:
HOLD-DOWN # 4 – w = 600 lb/ft

SECTION 7: MASONRY EXTERIOR WALLS – SHEAR WALL DESIGN

One-story residences constructed with exterior masonry walls and two-story residences with masonry walls at the first floor only shall have seismic designs and construction details as required by this section.

Engineering shall be provided for two-story dwellings with full-height exterior masonry walls.

7.1 Loads

All equivalent seismic weights shall be determined in the same manner as for wood frame construction except as modified by this section.

A. Actual weight of brick masonry shall be determined by multiplying thickness of wall in inches by 10 psf to obtain the weight per square foot of the wall.

B. Weight of concrete block shall be as shown in Table 3.2.

C. Equivalent seismic weight of masonry walls as they contribute load to the structure may be determined from Table 3.6.

7.2 Shear Walls

A. When exterior masonry walls are used, only masonry walls shall be considered as shear walls.

B. Minimum length for walls to be considered as shear walls shall be 2'-0".

7.3 Design of Shear Walls

A. Horizontal load capacity for walls of various effective lengths shall be determined from Figure 3.17 or 3.18 as applicable. Use the curve indicating the appropriate height of wall to make this determination. Determine total horizontal load capacity along a line of resistance in the same manner as for wood stud shear walls.

B. Overturning. Overturning resistance shall be determined as specified for wood stud shear walls except as modified herein.

1. Where steel lintels are used in lieu of reinforcing above window or door openings, consider the lintel to be part of the opening in determining opening height.

2. Use the "No Grade Beam Required" curve as shown on Figures 3.17, 3.18 and 3.19 to determine the horizontal load capacity of each wall. Add these capacities together along each line of resistance as described under Section 6.4 for wood frame shear wall design.

3. If horizontal capacity is insufficient as determined by 2. above, check

7.3B3 capacity as shown on the curve for the height of wall used and provide the grade beam indicated. (See Section 6.4D4 for grade beam installations.)

Alternative: As an alternative to 3. above, it may be desirable to grout the wall solid. When the "grouted solid" curve is referred to in Figures 3.17 or 3.19 and found sufficient, solid grouting of the wall may be used in lieu of a grade beam.

7.4 Masonry Wall Reinforcing (Zone 2 Only)

A. Reinforcing for 8" nominal masonry walls shall be as indicated on Detail 51/4 with reinforcing steel installed as shown on Details 51A/4 and 51B/4. Masonry lintels shall be constructed as shown on Details 51C/4 and 51D/4.

B. Where walls do not exceed 8'-8" in overall masonry height, partial reinforcing as shown on Detail 52/4 may be used. Reinforcing steel locations and lintel details shall be as shown on Details 52A/4 through 52D/4. Connections at the top of masonry walls shall be as shown in Details 53/4 through 57/4. These connections are adequate for walls up to 8'-8" in height as measured from first-floor line to top of wall.

7.4 Masonry Wall Reinforcing (Zone 3 Only)

A. Reinforcing for 8" nominal masonry walls shall be as indicated on Detail 51/4 with reinforcing steel installed as shown on Details 51A/4 and 51B/4. Masonry lintels shall be constructed as shown on Details 51C/4 and 51D/4.

B. Connections at top of masonry walls shall be as shown in Details 53/4 through 57/4. These connections are adequate for walls up to 8'-8" in height as measured from first floor line to top of wall.

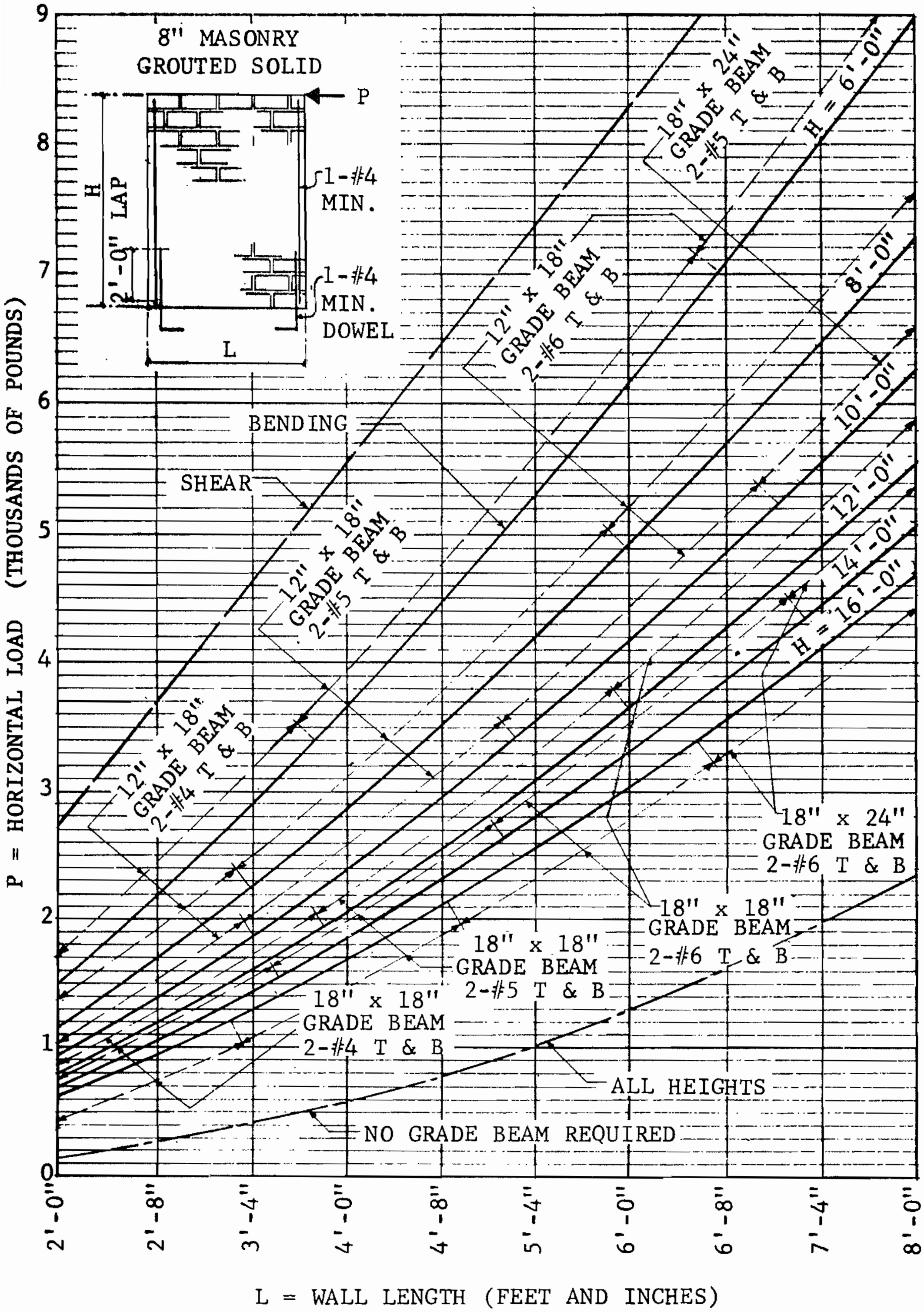


FIGURE 3.17 Horizontal Load Capacity for 8" Solid Grouted Masonry Walls.

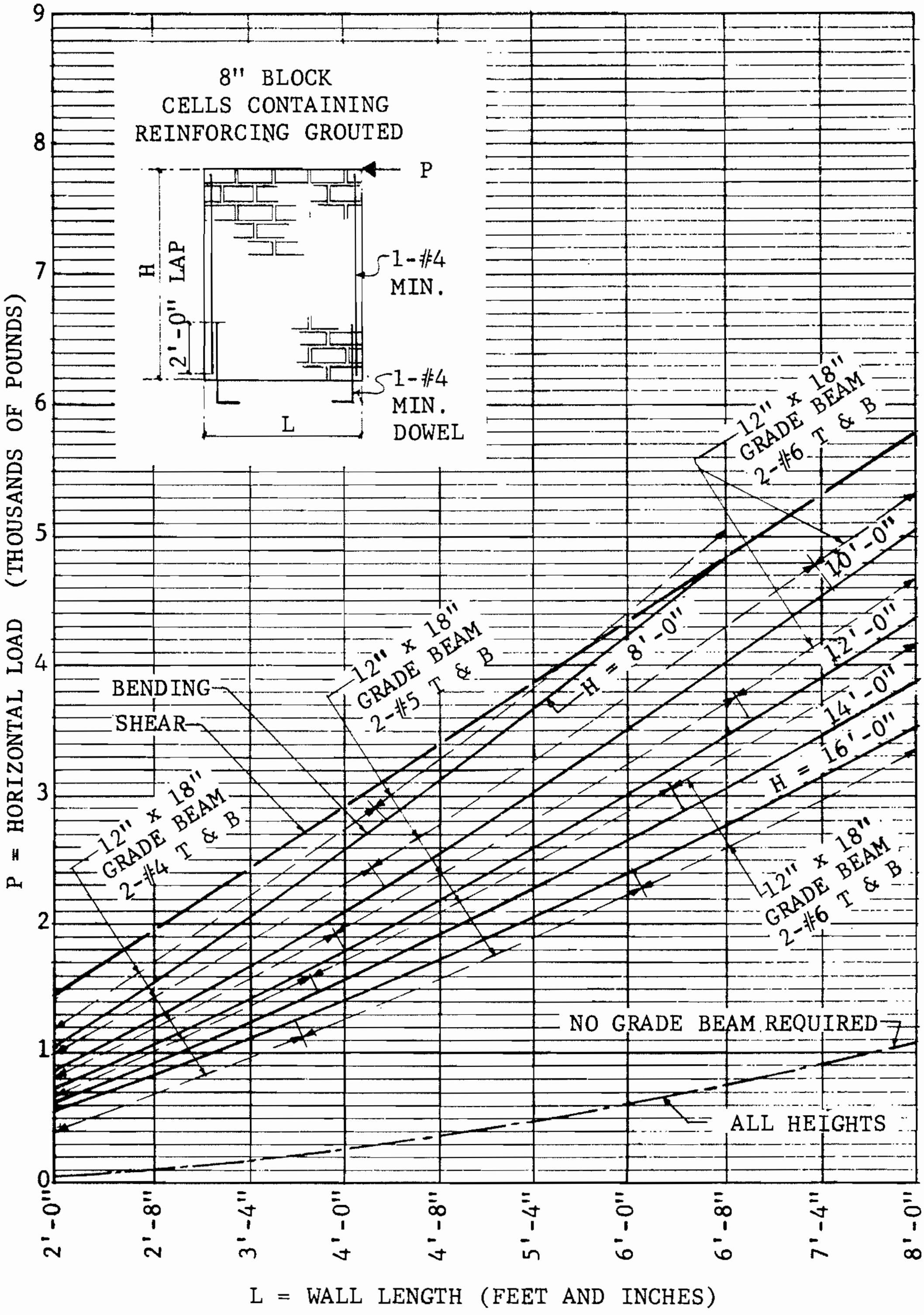


FIGURE 3.18 Horizontal Load Capacity for 8" Block Cells Containing Grouted Reinforcing.

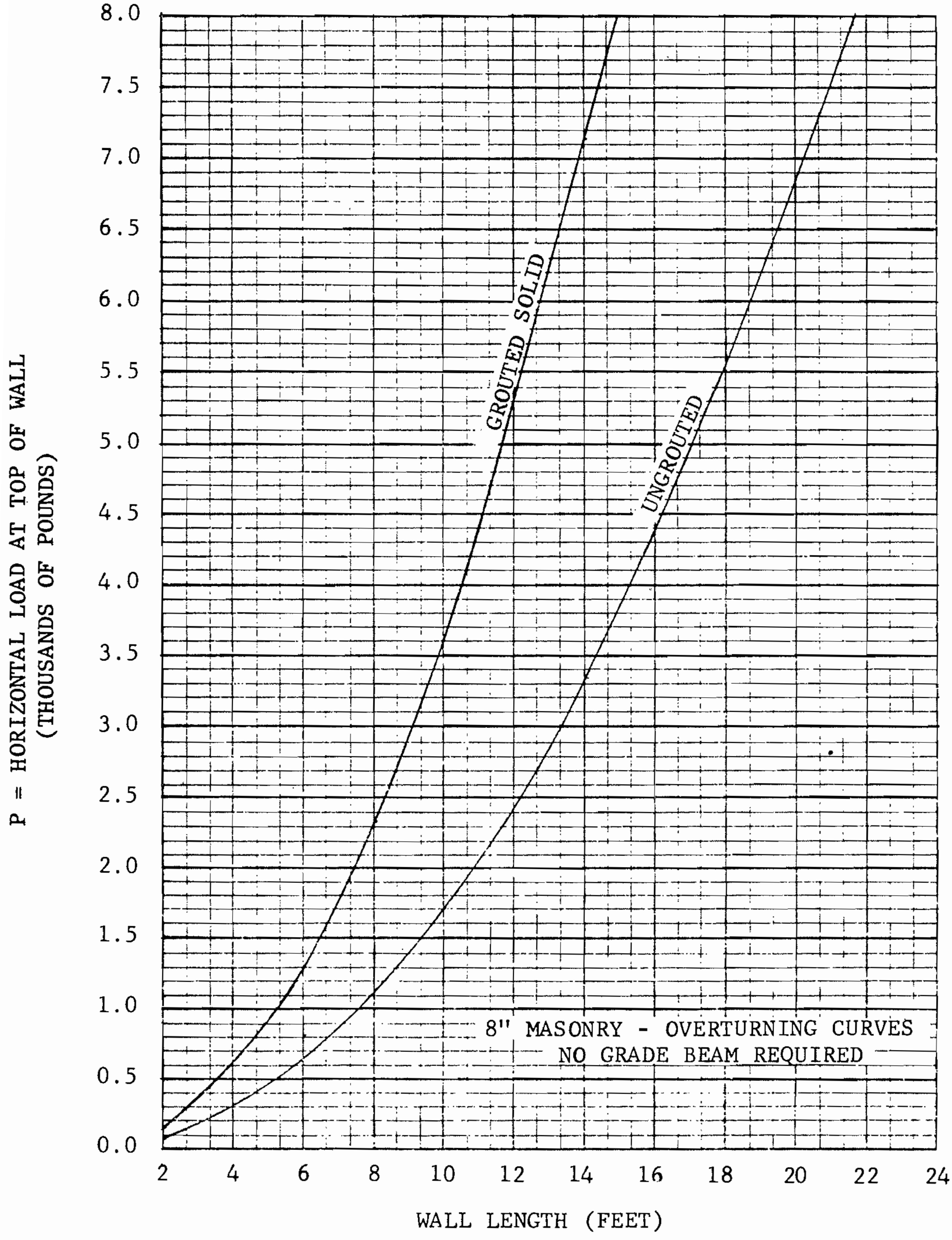


FIGURE 3.19 8" Masonry – Overturning Curve: No Grade Beam Required.

SECTION 8: MISCELLANEOUS STRUCTURAL CONSIDERATIONS

8.1 Considerations Applicable in Both Zone 2 and Zone 3

A. Roofs

1. Blocking at lines of support shall be 2" nominal width by the depth of the roof framing, and shall be installed and nailed as indicated on the Nailing Schedule.

2. In houses with flat roofs, sloped ceilings or other configurations requiring most interior walls to extend to the roof line:

   a. Spaced sheathing shall not be used.

   b. If straight sheathing is used, it is recommended but not required that 5/16" plywood be installed over the straight sheathing. Fastening of the plywood shall be with #14 or #15 gage staples by 1-3/8" minimum long, at 6" o.c. at edges of sheets and 12" o.c. to supports at the interior of sheets.

B. Floor Diaphragms

1. Plywood Sheathing

   a. Floor sheathing shall be half-inch or thicker plywood conforming to PS 1-74. Minimum size of sheets shall be 2'-0" by 4'-0". Joints shall be staggered as shown on the two right-hand figures in the lower portion of Table 3.10 or as indicated on the Errata Sheet for HUD-FHA Bulletin No. UM25d.

   b. Nailing shall be 10d common at 6" minimum o.c. at the edges of all sheets and at 10" o.c. to all intermediate supports.

   c. Plywood floor sheathing need not be blocked at interior abutments of sheets.

2. Sheathing Other Than Plywood

   Other floor sheathing used in lieu of plywood shall be approved through HUD Acceptance Procedures and shall be nailed such that a minimum shear of 250 pounds per foot is developed.

3. Glued Systems

   a. Where glued plywood floor systems are installed, adhesives conforming with the latest adopted or revised AFG-01 "Performance Specification for Adhesives for Field-Glueing Plywood to Wood Framing" shall be used. Glue shall be applied in accordance with manufacturer's recommendations.

8.1B3 b. Plywood shall be Underlayment T & G. Blocking at joints may be used in lieu of T & G plywood.

c. Nailing shall be 10d common nails at 12" o.c. to all bearings and at 6" o.c. at the edges of the diaphragm and over shear walls.

d. Plywood shall be installed with face grain perpendicular to the supports.

4. Openings

a. Where stairway or other openings occurring in second floor diaphragms are longer than 50% of the parallel dimension of the diaphragm at that location, a shear wall shall be provided below one of the long edges of the opening. Installation of such a shear wall is recommended even when such openings are less than 50% of the dimension of the diaphragm and are required (see below) when the opening is more than 4'-0" in width.

b. At stairways that require openings greater than 4'-0" in width, at other wide openings, and in split-entry homes, a shear wall shall be provided below at least one long edge of the opening. In split-entry homes the "long" edges shall be deemed to be those edges perpendicular to the adjacent exterior wall.

C. Chords

Chords consisting of exterior wall top plates (spliced as shown on Detail 22/4), beams or other horizontal framing members located directly under roof and second floor framing shall be continuous between intersecting cross walls. Where continuity cannot otherwise be attained (such as at a beam abutting the top plates), a splice shall be developed using a metal strap. Such splices shall be equivalent to Detail 22/4.

D. Sole Plates at Non-Designed Interior Walls

Sole plates at non-designed interior walls shall be connected to slabs on grade as shown on Details 19/4 or 20/4. Anchor bolts shall be 1/2" round at 6'-0" maximum o.c. and gun bolts as shown on Detail 20/4 shall be 2'-0" maximum o.c. When supported by wood framing, plates shall be nailed to the framing with two 16d nails to each perpendicular floor joist or as specified under Wall Framing in the Nailing Schedule for parallel joists.

E. Struts and Their Connections

As shown on Details 39/4 through 44/4, struts and their connections shall be used in the following circumstances:

1. When shear to the top plates, all or a portion of which are the top plates of the shear wall, exceeds 300 pounds per foot.
2. When a shear wall is totally offset from the diaphragm it supports or when only a portion of the shear wall is located under the diaphragm so that the top plates directly under the diaphragm have a shear in excess of 300 pounds per foot.

Examples:

a. Partial second floor with shear load at one end = 1500 pounds. Shear wall

8.1E2 contained mainly in one-story portion of house but extending under the second-floor diaphragm for a distance of 2'-0". 2 x 300 = 600 pounds. Strut load required = 1500 pounds — 600 pounds = 900 pounds.

b. Same condition as in a. except wall extends under diaphragm 2'-0" with 30" wide door adjacent and 3'-0" of additional wall on far side of door such that top plates under second-floor diaphragm are 7'-6" long. 7.5 x 300 = 2250 pounds allowed; no strut required for 1500-pound load.

F. Cutting and Notching

1. In exterior walls and bearing partitions, any wood stud may be cut or notched to a depth not exceeding 25% of the width of the stud.
2. In non-bearing partitions, any wood stud may be cut or notched to a depth not exceeding 40% of the width of the stud.

G. Bored Holes

1. A hole not greater in diameter than 1-1/2" in 2 x 4 stud, 2" in 2 x 6 stud, or 40% for other stud widths may be bored in any wood stud. Bored holes not greater than 60% of the width of the stud are permitted in non-bearing partitions or in any wall where each bored stud is doubled provided no more than two such successive doubled studs are so bored.
2. In no case shall the edge of the bored hole be nearer than 5/8" to the edge of the stud.
3. Bored holes shall not be located within 4" of the edge of a cut or notch.

H. Cripple Walls

Where cripple walls are used below first floor wood framing to enclose a crawl space only, shear-resisting material shall be provided continuously from the top to the bottom of such wall when it exceeds 14" in height. For walls less than 14" in height, solid blocking shall be used. When cripple walls are used in conjunction with knee wall construction or other circumstances wherein a full-story height occurs below the floor framing, the cripple wall shall be designed as a first-floor wall in a two-story residence.

I. Special Garage Front Wall Details

1. The Special Garage Front Wall Detail 45/4 shall be used on each side of the garage door for detached garages and attached garages when the garage protrudes from the main body of the house such that the wall containing the garage door opening is more than 6'-0" from a parallel shear wall. The detail shall not be used when a wall 4'-0" or wider is provided in line with the garage door. This detail is to be used to resist load from the garage only.
2. The Special Garage Front Wall Detail 46/4 shall be used at garage openings at the first floor of split-level construction. It shall also be used for garage walls in other two story houses when that portion of the house containing the garage protrudes from the main body of the house 6'-0" or more, such that no parallel shear wall occurs within 6'-0" of the garage door opening. Where protrusions are less than 6'-0" the second-floor diaphragm may be considered as cantilevering. The Special Garage Front Wall Detail shall be used to support only one-half

8.112 the area of the garage and the second floor immediately over. A cantilevered second floor may extend more than 4'-0" beyond the garage door opening or to one side of the garage.

a. When using this detail in conjunction with split-level designs, interior parallel walls shall be designed to support 100% of the seismic load from the two-story portion.

b. When using this detail in conjunction with other two-story designs, the rear wall of the garage shall support all seismic loads generated by the second floor walls and roof falling over the garage in addition to other tributary areas required.

3. The Special Garage Front Wall Details may not be used for shear walls 4'-0" in width or greater when such shear walls occur adjacent to the garage door opening. Walls qualifying as shear walls shall be designed as such and no increase need then be taken to the interior walls of the residence except as required by the multiplying factors (Section 3.3B3).

8.2 Considerations Applicable in Zone 2 Only.

A. Stud to Sole Plate Connections

Studs at exterior corners shall be fastened to the sill or sole plates with two BO framing anchors per stud as shown on Detail 24/4. One 1/2" round by 12" anchor bolt with standard washer shall be installed in each plate within 2-3/4" of the corner studs. This bolt shall be in addition to the other sill bolts required in the wall. Other framing anchors shown on Detail 24/4 need not be installed in Zone 2.

B. Split-Level Ties

1. Split-level ties as shown on Details 7/4 through 13/4 shall be used to tie framing of the mid-level roof to the second-floor walls in split-level construction and shall also be used at the lower roof of all houses with discontinuous roofs as defined by Section 2.3B. Equivalent ties may be provided as specified in subsection 8.2B6 below.

2. All wood frame structures of the types described in 1. above shall have the top plates of the mid-level (or "lower roof") exterior walls tied to the two-story (or higher roof) wall. Detail 12/4 shall be used for intersecting walls and Detail 13A/4 shall be used when the exterior walls of the two portions of the structure fall within the same plane.

3. When the roof framing of the mid-level (or lower) roof is perpendicular to the two-story (or higher roof) wall, ties shall be devloped as follows:

   a. When the line of intersection of the roof with the wall is horizontal, or nearly horizontal, rafters may be fastened directly to a stud in the wall at a maximum of 4'-0" o.c. as shown on the portion of Detail 7/4 titled "Rafter Adjacent to Stud."

   b. When roof slope is such that a proper nailed connection between the rafter and the stud cannot be developed, ties shall be made at either 4'-0" o.c. as shown on the portion of Detail 7/4 titled "Sloping roof—etc." or at not more than 16'-0" o.c. in the manner shown on Detail 9/4, but with blocking sloped as shown on Detail 11/4.

4. When the roof framing of the mid-level (or lower) roof is parallel to the two-story (or higher roof) wall, ties shall be developed as follows:

   a. Ridge poles shall be connected as shown on Detail 9/4. Intermediate ties and ties where no ridge pole is used shall be in accordance with Detail 11/4.

   b. Ties as detailed shall be provided at 16'-0" maximum o.c. except that structures containing no part of the living area (such as attached garages) may be tied at the exterior walls and ridge (or a point midway between the exterior walls) only.

   c. Ties shall be located midway between studs as shown on the details. Studs

8.2B4c to either side of each tie shall be fastened to top plates as shown on Detail 9/4 and to sole plates as shown on Detail 10/4.

5. Studs shall in all cases be continuous past the lower of the two roofs and shall extend to the plate line of the higher roof. For ties other than perpendicular framing ties at 4'-0" o.c., studs shall be 1700F_b to each side of the connection or shall be doubled as shown on Detail 13B/4. Where ties are made at 4'-0" o.c. no special connection need be made at the top and bottom of the stud.

6. Where connections other than those detailed are installed, the connections shall be designed for a horizontal load of 50 pounds per lineal foot in tension or compression at the line of juncture of the roof and the wall.

8.3 Considerations Applicable to Zone 3 Only

A. Stud to Sole Plate Connections

1. Studs at exterior corners shall be fastened to the sill or sole plates with two BO framing anchors per stud as shown on Detail 24/4. One 1/2" round by 12" anchor bolt with standard washer shall be installed in each plate within 2-3/4" of the corner studs. This bolt shall be in addition to the other sill bolts required in the wall.

2. The first two studs in each wall adjacent to exterior corners shall be fastened to the sole plate with a single BO framing anchor. One of these anchors shall be placed at the exterior of the structure and one at the interior.

3. Studs at the first story of two-story construction shall be fastened to the sole plate with BO framing anchors at 4'-0" o.c. in conjunction with all finishes other than plywood. Framing anchor placement shall be staggered from interior to exterior.

B. Split-Level Ties

1. Split-level ties as shown on Details 7/4 through 13/4 shall be used to tie framing of the mid-level roof to the second floor walls in split-level house construction and shall also be used at the lower roof of all houses with discontinuous roofs as defined by Section 2.3B. Equivalent ties may be provided as specified in subsection 8.3B6 below.

2. All wood frame structures of the types described in 1. above shall have the top plates of the mid-level (or "lower roof") exterior walls tied to the two-story (or higher roof) wall. Detail 12/4 shall be used for intersecting walls and Detail 13A/4 shall be used when the exterior walls of the two portions of the structure fall within the same plane.

3. When the roof framing of the mid-level (or lower) roof is perpendicular to the two-story (or higher roof) wall, ties shall be developed as follows:

   a. When the line of intersection of the roof with the wall is horizontal, or nearly horizontal, rafters may be fastened directly to a stud in the wall at a maximum of 4'-0" o.c. as shown on the portion of Detail 7/4 titled "Rafter Adjacent to Stud."

   b. When roof slope is such that a proper nailed connection between the rafter and the stud cannot be developed, ties shall be made at either 4'-0" o.c. as shown on the portion of Detail 7/4 titled "Sloping roof—etc." or at not more than 8'-0" o.c. in the manner shown on Detail 9/4, but with blocking sloped as shown on Detail 11/4.

4. When the roof framing of the mid-level (or lower) roof is parallel to the two-story (or higher roof) wall, ties shall be developed as follows:

   a. Ridge poles shall be connected as shown on Detail 9/4. Intermediate ties and ties where no ridge pole is used shall be in accordance with Detail 11/4.

b. Ties as detailed shall be provided at 8'-0" maximum o.c. except that structures containing no part of the living area (such as attached garages) may be tied at the exterior walls and ridge (or a point midway between the exterior walls) only.

c. Ties shall be located midway between studs as shown on the details. Studs to either side of each tie shall be fastened to top plates as shown on Detail 9/4 and to sole plates as shown on Detail 10/4.

5. Studs shall in all cases be continuous past the lower of the two roofs and shall extend to the plate line of the higher roof. For ties other than perpendicular framing ties at 4'-0" o.c., studs shall be 1700F_b to each side of the connection or shall be doubled as shown on Detail 13B/4. Where ties are made at 4'-0" o.c. no special connection need be made at the top and bottom of the stud.

6. Where connections other than those detailed are installed, the connections shall be designed for a horizontal load of 100 pounds per lineal foot in tension or compression at the line of juncture of the roof and the wall.

SECTION 9: MISCELLANEOUS NON-STRUCTURAL ITEMS

9.1 Glazing

A. A minimum of 1/8" clearance shall be provided between edge of window glass and the jamb on each side.

B. When 3/8" or thicker plywood shear panels are required it is recommended that 1/4" clearance be provided at all window edges along the line of resistance containing the panel(s).

C. When angle or strap-type hold-down anchors are used it is recommended that besides the clearances recommended or required above, additional clearance (in inches) be provided equal to 1.0 divided by two times the shortest length of wall (in feet) containing the hold-down anchors; e.g. 4' wall, additional clearance = 1/2x4 = 1/8".

9.2 Cabinets and Bookshelves

When wall or ceiling-hung cabinets, bookshelves or other similar installations are made, the top connection shall be developed with wood screws as shown on Detail 50/4. In addition, positive locking devices such as press-latches are recommended for doors.

9.3 Fireplaces and Chimneys

Masonry or concrete fireplaces and masonry, concrete or metal chimneys shall meet these minimum requirements of seismic resistance in addition to other requirements of the Minimum Property Standards and/or local ordinances.

A. Footings

Every masonry or concrete fireplace or chimney shall be supported upon concrete footings at least 12" thick extending at least 6" beyond the chimney or fireplace walls, and projecting at least 12" below the adjacent undisturbed natural ground surface. Where fireplace or chimney is located at an exterior wall, projection of the footing beyond the fireplace or chimney wall at the "exterior" side shall be 12" minimum. See Detail 47/4 in conjunction with this and other requirements relating to masonry fireplaces.

B. Veneer.

Veneer units shall be tied or anchored as required by Section 9.4 The required thickness of the chimney wall shall not include the thickness of veneer.

C. Thickness

1. Fireplace walls: Firebacks and jambs of masonry shall be not less than 8" nominal in thickness which may include 4" of firebrick.

9.3C 2. Chimneys: Every masonry or concrete chimney shall have solid walls at least 8" thick or shall have a flue lining surrounded by a layer of cement grout and concrete or masonry, the total thickness of which shall be not less than 4" outside the flue lining. The layer of grout shall be at least 1" in thickness.

D. Reinforcement

Every masonry or concrete fireplace and/or chimney shall be reinforced with steel consisting of the following:

1. Vertical reinforcement shall be #4 deformed bars hooked into the footing and spaced at not greater than 24" o.c. In chimneys of 40" or less, four of the vertical bars shall be continuous for the full height of the chimney and shall be hooked into the chimney cap. Such continuous bars shall be located as near to the corners of the chimney as practical and bends in the bars shall be avoided or minimized. Two additional vertical bars shall extend full-height for each additional flue in the chimney or for each additional 40" or fraction thereof of chimney width. Bars that are not required to be continuous shall extend from the footing to not less than 36" above the level of the smoke shelf.

   Exception: Fireplaces and chimneys constructed of hollow masonry units may have vertical reinforcing bars spliced to footing dowels, provided FHA inspection is made after reinforcing is placed and before cells are grouted.

2. #3 horizontal bars as ties looped around vertical bars and spaced at not more than 24" apart vertically from footing to chimney cap. A tie shall be provided at each bend in vertical bars.

3. Two 1/4" round bars looped around vertical steel in chimney cap and bond slabs.

4. Where units in hollow masonry construction are not bonded by overlapping of successive courses, one vertical 3/8" dowel shall be used to bond each masonry unit to the course above. This reinforcement shall be in addition to that required above.

E. Anchorage

All masonry and concrete chimneys shall be anchored at each floor or ceiling line more than 6'-0" above grade except where constructed completely within the exterior walls of the building. Anchorage shall consist of two 3/16" by 1" steel straps cast at least 18" into the chimney with a 180° bend with a 6" extension around the vertical reinforcing bars in the outer face of the chimney. Each strap shall be fastened to the structural framework of the building with two 1/2" bolts per strap. Where the joists do not head into the chimney the anchor straps shall be connected to 2" by 4" ties crossing a minimum of four joists. Such ties shall be connected to each joist with two 16d nails. Metal chimneys shall be anchored at each roof, floor and ceiling with two 1-1/2" by 1/8" metal strps looped around the outside of the chimney insulation and nailed with six 8d nails per strap to the roof or ceiling framing. Each strap shall have a length of not less than 12" between the metal chimney and any combustible material to which it is attached.

9.3 F. Cutting of Plates

Where plates are cut to permit the passage of the chimney, 3/16" by 1" steel strap anchors shall be hooked into the concrete bond beam and secured to the end of each plate with at least two 1/2" by 4" lag screws or two 1/2" bolts.

G. Lintels

Brick work over the fireplace opening shall be supported by a steel angle not smaller than 3" by 2-1/2" by 3/16" set with not less than 3" of end bearing on the masonry or by a reinforced masonry or concrete bond beam.

9.4 Veneer

A. Anchored Veneer

1. Anchored veneer height may not exceed 25'-0" at plate line and 28'-0" at gables. Veneer may be no less than 3/4" nor more than 5" nominal thickness.

2. Masonry or stone veneer not exceeding 4'-0" in height shall be anchored as herein required except that ties need only be provided 6" below the top of the veneer at 16" maximum horizontal spacing.

3. Anchored veneer not designed in accordance with these requirements shall meet HUD Acceptance Procedures and its attachment shall be designed to resist a horizontal force equal to two times the weight of the veneer.

4. Veneer shall support no load other than its own weight and the vertical dead load of the veneer above.

5. Lintels. When veneer is not self-supporting an incombustible and corrosion-resistant lintel shall be provided.

   a. Where openings are less than 5'-0" in width the lintel may rest upon the veneer jamb.

   b. Where veneer above lintel does not exceed 5'-0" in height the lintel may be bolted to the wood framing.

   c. For openings greater than 5'-0" in width with veneer above lintel more than 5'-0" in height, lintels shall be supported on a column built into the jamb.

6. Installation to masonry or wood with ties.

   a. All veneer ties shall be corrosion-resistant metal capable of resisting in tension or compression a force equal to two times the weight of the attached veneer. In other than masonry construction the ties shall be anchored to the wall framing. If made of sheet metal, veneer ties shall

9.4A6a be not smaller in area than 1/16" by 1" or if made of wire, not smaller in diameter than No. 9 gage wire. Anchor ties shall be spaced so as to support not more than two square feet of wall area, but not more than 24" o.c. horizontally. In Seismic Zone 3, anchor ties at masonry walls shall be connected to horizontal joint reinforcement of No. 9 gage or equivalent. The joint reinforcement shall be continuous with butt splices between ties permitted.

b. For masonry veneer units the ties shall engage by a 180° bend a continuous horizontal reinforcement wire of No. 8 gage laid on the center line of the veneer in the mortar of a stretcher joint.

c. For stone slab veneer, the ties shall engage drilled eyes of corrosion-resistant metal dowels located on the center line of the edges of the units and not farther apart than 24" around the periphery of each unit with not less than four ties per veneer unit. If not tight-fitting, the holes for dowels may be drilled not more than 1/16" larger in diameter than the dowel, with the hole countersunk to a diameter and depth equal to twice the diameter of the pin to provide a tight-fitting key of cement mortar at the pin locations when the mortar in the joint has set.

d. When applied over wood stud construction, an approved paper shall first be applied over the sheathing or wire between studs and grout shall be poured into the 1" space between facing and paper.

7. As an alternative for wood stud construction, veneer may be applied with 1" minimum grouted backing space which is reinforced by not less than 2" by 2" No. 16 gage galvanized wire mesh placed over waterproof paper backing and anchored directly to stud construction. The galvanized wire mesh shall be anchored to wood studs by galvanized steel wire furring nails at 4" o.c. or by barbed galvanized nails at 6" o.c. with a 1-1/8" minimum penetration. If this method is applied over solid sheathing, the mesh must be furred for embedment in grout. The wire mesh must be attached at the top and bottom with not less than 8d common wire nails. The grout fill shall be placed to fill the space intimately around the mesh and veneer facing.

B. Adhered Veneer

1. Adhered (adhesive) veneer height shall not exceed 25'-0" at plate line and 28'-0" at gables. Veneer thickness shall be not less than 1/8" and not more than 1". Veneer shall not weigh more than 15 psf maximum.

2. Where veneer is designed rather than meeting the arbitrary requirements contained herein the veneer and its backing shall have a bond to the supporting element sufficient to withstand a shearing stress of 50 pounds per square inch under test.

3. Backing for exterior veneer shall provide a weather-resistant barrier. Exterior veneer including its backing shall form a weatherproof covering.

4. One of the following methods of application may be used:

a. A paste of neat Portland Cement shall be brushed on the backing and the

9.4B4a back of the veneer unit. Type M or S mortar then shall be applied to the backing and the veneer unit. Sufficient mortar shall be used to create a slight excess to be forced out the edges of the units. The units shall be tapped into place so as to fill the space between the units and the backing completely. The resulting thickness of mortar in back of the units shall be not less than 1/2" nor more than 1-1/4".

b. Units of masonry or stone shall be restricted to 81 square inches in area unless the back side of each unit is ground or box-screeded to true up any deviations from plane. These units may be adhered by means of Portland Cement. Backing may be of masonry, concrete or Portland Cement plaster on metal lath. Type M or S mortar shall be applied to the backing as a setting bed. The setting bed shall be a minimum of 3/8" thick and a maximum of 3/4" thick. A paste of neat Portland Cement or half Portland Cement and half graded sand shall be applied to the back of exterior veneer units and to the setting bed and the veneer pressed and tapped into place to provide complete coverage of the mortar bed and the veneer unit. A Portland Cement grout shall be used to point the veneer.

C. Grout

Grout used in conjunction with ties or paper backing for anchored veneer may be considered to have the same shear resistance value as for exterior stucco. Shear resistance of adhered veneer shall be the resistance of the backing installed. As is true of all other exterior finishes, interior finish may be used to develop shear resistance in lieu of exterior finish, but both finishes shall be nailed as specified elsewhere herein.

9.5 Mechanical Equipment

A. Water Heaters. Water heaters shall be installed in accordance with Detail 49/4 or in an equivalent manner. Water heaters shall be attached at their tops to resist a minimum force of 80 pounds applied in any direction. Nailing or other mechanical fastening shall be made at the base to resist the same horizontal load. These requirements shall apply to all water heaters whose height is greater than the least dimension (or diameter) of the body of the heater.

B. Floor Furnaces. Floor furnaces with blower motors and burner units located 3'-0" or less above the base of the unit shall be mechanically fastened at the base only to prevent sliding. Units with heavy portions of their construction at an elevation higher than 3'-0" shall be tied to the adjacent structure at or near the top to prevent overturning.

C. Other mechanical equipment such as air conditioners, roof coolers, etc., shall be mechanically fastened to the supporting structure to prevent sliding and, where applicable, overturning. The equipment itself shall have all major substructures similarly connected.

D. Gas Valves. Gas shut-off valves shall be provided for each individual residence and shall be labeled as such. They shall be located for easy access by the homeowner.

PART IV CONSTRUCTION DETAILS

COMMENTARY ON DETAILS

Wood frame construction techniques are so standardized that, customarily, very few details are provided in conjunction with dwelling plans. Also customarily, and quite logically, communities develop construction procedures consistent with their experience and special problems. Thus "standardized" construction procedures for a particular area may be at considerable variance with procedures elsewhere. Predesigned details presented in this chapter are intended to develop a shear path and to mitigate other hazards created by earthquakes. Many of the details also will improve the wind resistance of dwelling construction. The details, and the notes on them, are not intended to supplant local standardized procedures, but to supplement them with requirements designed to improve seismic resistance. It is intended that the details be modified as necessary to attain the required results when local procedures vary from the construction methods indicated. It should be emphasized that it is essential to satisfy the requirements implied by a detail, not necessarily the entire "picture" shown on the detail.

In addition to modifications to existing construction techniques, new details are presented which achieve greater strength and/or safety for elements that have proven to be insufficient in performance. These include, for example, the special garage front wall details, the cabinet detail, the water heater detail, plus the requirements stipulated but not detailed in the Design Methodology for other mechanical equipment as well as for framing and finishing.

Although FHA inspection procedures will assure the implementation of the recommended revisions to construction procedures, satisfactory achievement of the desired results will lie largely with the contractor and his subcontractors. An important part of this chapter is the commentary on each detail, intended to familiarize all concerned as much as possible with the intent behind the details.

Because few contractors, including those building homes in areas of high earthquake probability and intensity, are familiar with principles of seismic design it is recommended that they, particularly, read Chapter II—1 to facilitate understanding of the purposes of the details. Chapter II—1 also supplies definitions of terms used in discussing the tables and predesigned details.

Tables and Details. Most of the tables and details presented in Part IV are applicable in either seismic Zone 2 or Zone 3. The details are numbered consecutively and, in most cases, "picture" the same conditions whether they apply to Zone 2 or 3. Considerable care must be taken to assure the correct detail is used for design and construction in the appropriate zone.

Table 4.1 serves as a **Table of Contents for the Details**, identifying their applicability based on type of exterior wall construction and on the configuration of the dwelling. This table should be especially useful to home designers and plan-checkers in the preliminary determination of which details will be required for a particular structure. During final design the need for each detail, as well as the determination that a special section or detail not presented herein is needed, should be based on the conditions developed by the individual design. Following Table 4.1 comes commentary on the details, then the details, then tables.

Table 4.2. The length of the top and bottom **Reinforcing in Grade Beams** beyond the end of a shear wall is tabulated in Detail 31/4 for use in conjunction with Details 29/4 through 31/4.

Table 4.3: **Allowable Shear.** Already presented as Table 3.9, this table is repeated here for use by the contractor in determining required methods of attachment of various shear-resisting materials. All fastenings are based on wall construction using 2 x 4 studs at 16" o.c. This minimum size and spacing shall be adhered to for all exterior walls and other designated shear walls. Finish material shall be attached to all other walls in the same manner; the studs may be smaller or the spacing greater for interior, nonstructural walls.

Table 4.4: Plywood Allowable Shears, reprinted courtesy the American Plywood Association, lists values for plywood siding when it is to be the finish material or on the occasions when plywood sheathing is required because the finish material being used is incapable of resisting the seismic loads by itself. Again, this table may be of more use to the designer than to the man in the field but it lists the minimum nailing required for installation of each particular thickness of plywood. Generally speaking, nailing of plywood other than siding should be noted on the plans. Except when otherwise indicated plywood siding nailing may be of the nail size indicated at 6" o.c. to the edges of all sheets and 12" o.c. to intermediate supports. Arrangement of the sheets may be as shown in drawings at the bottom of the table. The sheet arrangements at the right of the table shall also be used when applying plywood flooring or subflooring.

Table 4.5: Nailing Schedule. This table, a key to obtaining a proper shear path in house construction, is presented twice: Once for use in Zone 2 and once for use in Zone 3. Again, care must be taken to assure use of the correct Nailing Schedule for the appropriate zone. Some nailing requirements are specified on certain of the details. (Where they are more severe than those of Table 4.5, local requirements should be followed.) Nailing schedules predicated on seismic loads include: (1) Nailing of blocking to top plates at roofs and floors; (2) nailing of end rafters, trusses and joists; and (3) sole plate nailing. In addition, nailing for those items as shown in Zone 2 Nailing Schedule may be considered adequate in 15psf wind zones. The Zone 3 "normal" nailing may be used for up to 30psf wind designs for one-story construction and 20psf for two-story construction. "Lightweight" Zone 3 nailing as specified at the center of the second page of the schedule may be considered adequate for 25psf wind for one-story construction and 15psf for two-story. These statements are based on a 2 to 1 diaphragm ratio for one-story houses and 1-1/2 to 1 for two-story. Allowable wind loads may be increased proportionally as diaphragm ratio is decreased from those above-stated ratios.

TABLE 4.1 LISTING AND USE OF DETAILS

Notations

R = Virtually always required. WA = Use where applicable.
( ) = Detail No. in parentheses may be applicable instead.
Br roof = Use detail if roof is broken. Z2 and Z3 = Zone 2, Zone 3.

DETAILS No.	Description	EXTERIOR WALL CONSTRUCTION — WOOD FRAME: One Story	WOOD FRAME: Two Story	WOOD FRAME: Split Level	MASONRY (Masonry 1st, Wood Frame 2d): One Story	MASONRY: Two Story	MASONRY: Split Level	Page IV—
1	Exterior bearing walls at roof	R	R	R		R	R	19
2	Exterior end walls at roof	R	R	R		R	R	20
3	Exterior walls at second floor		R	R				21
4	Exterior walls — second floor overhang		WA	WA		•WA similar	WA similar	22
5	Interior shear walls	WA	WA	WA		WA	WA	23
6	Bracing at corners of openings	optional	optional	optional				24
7	Shear transfer: Split-level tie (rafters perpendicular)	Br roof	Br roof	R	Br roof	Br roof	R	24
8	Use of split-level ties (rafters parallel)	Br roof	Br roof	R	Br roof	Br roof	R	25
9	Split-level ties at ridge pole	Br roof	Br roof	R(11)	Br roof	Br roof	R(11)	26
10	Split-level ties at base of stud	Br roof	Br roof	R	Br roof	Br roof	R	26
11	Split-level ties — intermediate	Br roof	Br roof	Z3-R Z2-R(9)	Br roof	Br roof	Z3-R Z2-R(9)	27
12	Split-level ties at mid-level top plates	Br roof	Br roof	R	Br roof*	Br roof*	R*	27
13	Split-level ties — continuous exterior walls and use of double studs	Br roof	Br roof	WA	Br roof	Br roof	WA	28

*Nailer at masonry wall in lieu of top plate. **Except where full basement occurs.

Table 4.1 continued

TABLE 4.1 LISTING AND USE OF DETAILS (continued)

Notations

R = Virtually always required. WA = Use where applicable.
( ) = Detail No. in parentheses may be applicable instead.
Br roof = Use detail if roof is broken. Z2 and Z3 = Zone 2, Zone 3.

DETAILS		EXTERIOR WALL CONSTRUCTION						
		WOOD FRAME			MASONRY (Masonry 1st, Wood Frame 2d)			
No.	Description	One Story	Two Story	Split Level	One Story	Two Story	Split Level	Page IV–
14	Stem wall footings	R(15)**	R(15)**	R(15)**				29
15	"Trench" footings	R(14)	R(14)	R(14)				29
16	Basement walls	WA	WA	WA			R	31
17	Retaining wall at split-level			R			R	32
18	Knee wall construction	WA	WA	WA				32
19	Anchor bolts at interior non-bearing walls (For slab on grade construction)	R(20)	R(20)	R(20)	R(20)	R(20)	R(20)	33
20	Gunbolts at interior non-bearing walls (For slab on grade construction)	R(19)	R(19)	R(19)	R(19)	R(19)	R(19)	33
21	Interior shear wall to wood floor framing (For wood floor construction)	WA	WA	WA	WA	WA	WA	34
22	Chord splice	R	R	R		R	R	34
23	Standard framing anchor use	R	R	R	R	R	R	35
24	Fastening studs to sole plates	R	R	R				36
25	Typical strap hold-down (Capacity = 1700 lbs)	WA	WA	WA		WA	WA	37
26	Framing Anchor hold-down (Capacity = 800 lbs)	WA	WA	WA		WA	WA	37

*Nailer at masonry wall in lieu of top plate. **Except where full basement occurs.

Table 4.1 continued

TABLE 4.1 LISTING AND USE OF DETAILS (continued)

Notations

R = Virtually always required. WA = Use where applicable.
( ) = Detail No. in parentheses may be applicable instead.
Br roof = Use detail if roof is broken. Z2 and Z3 = Zone 2, Zone 3.

DETAILS		EXTERIOR WALL CONSTRUCTION						
		WOOD FRAME			MASONRY (Masonry 1st, Wood Frame 2d)			
No.	Description	One Story	Two Story	Split Level	One Story	Two Story	Split Level	Page IV–
27	Second floor strap hold-down (Capacity = 800 lbs)		WA	WA				37
28	Typical shear wall installation	R	R	R		R	R	38
29	Typical grade beam at shear wall (hold-downs)	WA	WA	WA	WA	WA	WA	39
30	Typical grade beam and shear wall at corner(hold-down)	WA	WA	WA	WA	WA	WA	40
31	Length of reinforcing	WA	WA	WA	WA	WA	WA	41
32	12" x 18" grade beams	WA	WA	WA	WA	WA	WA	42
33	18" x 24" grade beams	WA	WA	WA	WA	WA	WA	42
34	Installation of angle hold-down anchors	WA	WA	WA		WA	WA	43
35	Strap hold-down at floor framing		WA	WA				44
36	Strap hold-down at beam or header		WA	WA				44
37	Installation of hold-down at floor framing		WA	WA				44
38	Installation of hold-down to beam or header		WA	WA				44
39	Strut – Capacity 1200 lbs	WA	WA	WA	WA*	WA*	WA*	45

*Nailer at masonry wall in lieu of top plate. **Except where full basement occurs.

Table 4.1 continued

TABLE 4.1 LISTING AND USE OF DETAILS (continued)

Notations

R = Virtually always required. WA = Use where applicable.
( ) = Detail No. in parentheses may be applicable instead.
Br roof = Use detail if roof is broken. Z2 and Z3 = Zone 2, Zone 3.

DETAILS		EXTERIOR WALL CONSTRUCTION						
		WOOD FRAME			MASONRY (Masonry 1st, Wood Frame 2d)			
No.	Description	One Story	Two Story	Split Level	One Story	Two Story	Split Level	Page IV–
40	Strut – Capacity 600 lbs	WA	WA	WA	WA*	WA*	WA*	45
41	Strut – Capacity 2400 lbs	WA	WA	WA	WA*	WA*	WA*	46
42	Strut connection – rafters perpendicular (Capacity = 600 lbs)	WA	WA	WA	WA*	WA*	WA*	47
43	Strut connection – rafters perpendicular (Capacity = 1200 lbs)	WA	WA	WA	WA*	WA*	WA*	47
44	Strut connection – rafters perpendicular (Capacity = 2400 lbs)	WA	WA	WA	WA*	WA*	WA*	48
45	Special garage front wall detail For single-story garages	WA						49
46	Special garage front wall detail For garages with second floor above		WA	R				51
47	Typical fireplace construction	WA	WA	WA	WA	WA	WA	53
48	Chimney tie at 2nd floor		WA	WA		WA	WA	54
49	Typical water heater installation	R	R	R	R	R	R	55
50	Cabinet fastenings	R	R	R	R	R	R	56
51	Masonry wall – fully reinforced				Z3-R Z2-R(52)	Z3-R Z2-R(52)	Z3-R Z2-R(52)	57
52	Masonry wall – partially reinforced (Zone 2 only)				Z2-R(51)	Z2-R(51)	Z2-R(51)	59

*Nailer at masonry wall in lieu of top plate. **Except where full basement occurs.

Table 4.1 continued

TABLE 4.1 LISTING AND USE OF DETAILS (concluded)

Notations

R = Virtually always required. WA = Use where applicable.
( ) = Detail No. in parentheses may be applicable instead.
Br roof = Use detail if roof is broken. Z2 and Z3 = Zone 2, Zone 3.

DETAILS		EXTERIOR WALL CONSTRUCTION						
		WOOD FRAME			MASONRY (Masonry 1st, Wood Frame 2d)			
No.	Description	One Story	Two Story	Split Level	One Story	Two Story	Split Level	Page IV–
53	Exterior masonry bearing wall				R		R	61
54	Exterior masonry end wall				R(55)		R(55)	63
55	Exterior masonry gable end wall				R(54)		R(54)	65
56	Exterior masonry bearing wall supporting second floor					R	R	67
57	Exterior masonry non-bearing wall supporting second floor					R	R	67
58	Footing — floor joists bearing (For wood floor construction)				R	R	R	69
59	Footing — floor joists parallel (For wood floor construction)				R	R	R	69
60	"Trench" footing and slab on grade (For slab on grade construction)				R	R	R	69

*Nailer at masonry wall in lieu of top plate. *Except where full basement occurs.

DETAILS AND COMMENTARY

Note: The 60 details presented starting on Page IV–19 may be removed and duplicated on an office copier for inclusion with plans. In a few cases blank spaces have been left on the details to be filled in as required by the individual job. The details should be referenced on the plans.

Detail 1/4 Exterior Bearing Walls at Roof

This detail provides a shear path from the roof sheathing through the blocking to the top plate and to the shear-resisting material. The method of framing may vary, but the transfer of horizontal loads from the sheathing down to the finish (shear-resisting) material must be developed.

An example of variance in construction procedures, as noted in the Introduction, is indicated in Detail 1D. Prefabricated roof trusses are frequently fabricated in manners other than the method shown. It is not the intent of this detail to indicate a required method of roof truss fabrication but to spell out the shear transfer details.

Detail 2/4 Exterior End Walls at Roof

This detail is similar to the details discussed above and the same comments concerning shear transfer apply here. The major difference is that transfer of shear takes place in the end rafter or truss rather than through blocking. One major distinction between this and Detail 1 is that for the application of shear-resisting material where such material is applied in sheets, it is required that the sheets be spliced at the top plates to effect direct shear transfer. This requires that the finish material applied above the top plate extend down approximately 3/4" below the bottom of the framing member or truss.

Detail 3/4 Exterior Walls at Second Floor

This detail shows continuous shear-resisting materials and finishes interrrupted at the second floor line. It is of little consequence how the rough framing is hidden from view in Details 3B and 3D. The importance of Detail 3 is essentially the same as in the previous detail – effecting shear transfer from the upper wall and the floor sheathing to the first-floor wall.

The Nailing Schedule footnote states that certain nailing may be reduced by 50% when the shear-resisting material is continuous, as shown on Details 3A and 3C. When shear-resisting material on the exterior of the building is uninterrupted it transfers the shear directly downward past the floor framing and shear-transfer connections therefore need not be as heavy. To achieve uninterrupted shear resistance with sheet-type exterior finishes, the material must be spliced to either the first-floor top plate or the sill plate at the second floor line. Since shear materials must be a minimum of 2'-0" x 4'-0" a splice at both these locations would not be acceptable. Nine- or ten-foot-long sheets are therefore normally used to attain the desired continuity.

Detail 4/4 Exterior Walls—Second Floor Overhang

When the second floor overhangs the first floor exterior wall it is necessary to place shear transfer blocking beneath the second-floor wall and also above the first-floor wall. A similar condition exists when floor joists are parallel to the wall. How the double floor joists in Details 4C and 4D are supported is not indicated; it is assumed that the joists span to cantilever beams. In the cases illustrated, the shear from the second-floor walls is transferred into the second-floor sheathing and to the blocking or end joists carrying the load down to the top plate of the first-story wall.

Detail 5/4 Interior Shear Walls

When interior walls extend to the roof framing, they should virtually always be considered as a shear-resisting element. Exceptions to this rule are walls less than 4'-0" in length and relatively short walls in houses where all interior walls extend to the roof. Although not shown, the connection of parallel trusses to interior shear walls should be made in the same manner as shown on Detail 5B. Since the bottom chord of the trusses is considered ceiling framing, walls extending to the bottom of the truss need not be considered as shear walls unless specifically indicated on the plans. It would be impractical (although not impossible), for instance, to require that shear wall connections to perpendicular trusses be made in a manner similar to Detail 5A. In Detail 5B, however, nailing of the sheathing can be made to the top chord of the truss while the nailing to the top plate would obviously be developed from the bottom chord. It is assumed that the truss is capable of transferring the load from the top to the bottom chord. This virtually always proves to be true:

Detail 6/4 Bracing at Corners of Openings (Optional)

Although certain walls containing openings may be considered as "solid" when considering overturning, damage to finish at the corners of windows and doors may be expected in an earthquake. In terms of the overall integrity of the structure such damage may be relatively slight but it is unsightly, at the least. In the case of severe cracking or other damage (depending on type of finish), repair may be quite costly, involving not only patching or replacement but repainting of the entire structure. Detail 6/4 attempts to minimize damage at the corners of openings. Although such damage is common, it is not considered major and the detail is therefore optional.

Detail 7/4 Shear Transfer at Broken Roofs/Split-Level Tie With Rafters Perpendicular

Details 9, 11 and 12 show the way a mid-level roof should be tied to the two-story portion of a split-level house and should also be used for other similar discontinuous-roof conditions. In order to depict the tie clearly, the shear transfer from the end rafter to continuous solid blocking in the stud wall is not completely shown on Details 9 and 12 and not called out on Detail 11. This detail indicates that 2-16d nails should be used to connect the end rafter (or blocking) to the blocking in the wall and that the interior finish should also be nailed to the wall blocking.

Occasionally, the mid-level roof is framed perpendicular to the two-story wall. In addition, one-story houses with broken roofs are frequently framed as shown on Detail 7. In these cases, the tie between roof levels should be made in one of the two manners shown.

Details 8/4 through 13/4   Split-Level Ties

Pictures in Part I provide ample evidence that split-level homes tend to separate at the break in levels when the mid-level roof is framed parallel to the two-story wall. Practice has been to nail the end rafter to each stud or, worse, to nail only intermittently to the studs. Details 8/4 through 13/4 indicate required ties to be placed in such construction.

Detail 8/4, an isometric view of the joining of the mid-level roof to the two-story portion of the house, is an informational detail to picture more clearly the way the ties are to be used. The connections themselves are shown on the referenced details.

The intent of Details 9, 11 and 12 combined is to provide ties at the mid-level front and rear walls, at the ridge, and at 8'-0" maximum o.c. in Zone 3 and 16'-0" maximum o.c. in Zone 2.

Where standard framing is used, the ridge pole should be connected to the second-floor wall as indicated on Detail 9/4. Note that a stud should be placed approximately eight inches either side of the ridge pole; a single stud adjacent to the ridge pole would be too weak in bending to take the tension or compression load developed. When a prefabricated truss, or other method of framing not requiring a continuous ridge pole, is used, Detail 11/4 should be substituted to achieve the ridge connection.

The top plates of the mid-level walls intersecting the two-story walls should be connected as shown on Detail 12/4. Again, studs should be placed eight inches each side of these plates. Obviously, additional studs will be placed below the top plates as the end framing of the mid-level wall.

The horizontal distance between the mid-level exterior top plates and the roof ridge is normally more than 8'-0". Detail 11/4 provides an intermediate connection which should be used so that ties are provided at 8'-0" maximum o.c. in Zone 3 and 16'-0" maximum o.c. in Zone 2. A stud must be placed eight inches each side of each tie and a special connection is necessary at the top and bottom of these studs. The top connection is shown on Detail 9/4 and the connection to the sole plate is shown on Detail 10/4. The studs must have a minimum bending strength of 1,700 psi. Where it is desired to use the same stud material as is used for the remainder of the job, the studs each side of each connection should be doubled and installed as shown on Detail 13B/4.

Occasionally the front or rear wall of the mid-level is in line (in plan view) with the parallel exterior wall of the two-story portion. When this occurs, Detail 13A/4 should be used instead of Detail 12/4 for the connection of the mid-level top plate to the two-story portion.

Although presented for use in split-level construction, these details should be used in any case of a vertical break in roof framing — whenever roof sheathing cannot be applied continuously.

When framing of the mid-level roof is perpendicular to the two-story wall, Detail 7/4 should be used. This detail does not require use of special studs because the connections are closer together.

If alternative details to the predesigned details are used, they must be designed for a minimum of 100 pounds-per-foot (tension or compression) of interface between the two portions of the structure in Zone 3 and one-half that amount in Zone 2.

Detail 14/4 Stem Wall Footings (Zone 2 and Zone 3)

The important objective of Detail 14/4 is the realization of the required shear-transfer, including proper size and spacing of sill bolts to the foundation. The required size and spacing has been omitted in the Detail and should be filled in by the preparer of the plans in accordance with the information developed through the use of the Calc Forms. These bolts should never be less than 1/2" round at 6'-0" o.c., however. (Such considerations as size, thickness or depth of footing and provision for proper crawl space are not incorporated in the detail; some are discussed, however, in Part II, "Foundations.")

Detail 14/4 is the first of several that are printed in two forms, once on yellow paper for use in Zone 2 and once on pink paper for use in Zone 3. Care should be taken in the designation of the proper detail.

Detail 15/4 "Trench" Footings (Zone 2 and Zone 3)

The three details shown in Detail 15/4 serve two purposes:

First, the required method for the installation of sill bolts is shown. The requirement that the bolts extend 4" minimum into the footing proper is uniformly applicable.

Secondly, floor slabs not poured monolithically with footings are apt to separate and move during seismic disturbances. This is particularly true when a waterproof membrane has been extended between the top of the footing and the bottom of the slab. The contractor is afforded the option of installing either dowels or a continuous key to improve seismic resistance for this condition. Although shown near the center of the footing, the dowels may be placed in line with the anchor bolts on Detail 15B and may also be placed in the same manner on Detail 15A provided the horizontal leg of the dowel is lengthened to extend a minimum of 15 inches into the slab. Because of the lighter loading the installation of the dowels or key have been made optional in Zone 2 but are required for Zone 3.

The waterproof membrane shown on these details indicates the recommended method of installation where it is required that the membrane be extended over the top of the footing. It must not be inferred that such a membrane is required for seismic reasons, nor that the membrane must be extended across the footing. Such requirements, where made, are determined by local HUD offices to meet other conditions.

The same remarks with regard to footing size and anchor bolt size and spacing made for Detail 14/4 apply to Detail 15/4 as well.

Detail 16/4 Basement Walls

In addition to providing shear-transfer requirements for seismic forces this detail also provides for the required reaction for nominal retaining wall action (30 lb/ft^2/ft equivalent fluid pressure, 7'-6" maximum earth height). (This is discussed in Part II, "Basements.")

The same remarks with regard to footing size and anchor bolt size and spacing made for Detail 14/4 apply to Detail 16/4 as well.

Details 17/4 and 18/4 Retaining Walls at Split-Level Construction and Knee Walls

These details, again, indicate the shear-transfer requirements only. The finish materials themselves are not shown but should be applied at the second-floor line in accordance with Detail 3 and at the base as shown on Detail 14.

In some cases the mudsill is placed on the top of the retaining wall and the sole plate connected to it. In this instance the nailing of the sole plate should be in accordance with Table 4.5. Wherever possible, the finish material should be attached directly to the mudsill. The sole plate nailing may then be reduced in accordance with the footnote in Table 4.5. Although the alternate detail is indicated in conjunction with knee wall construction, the same remarks apply any time a similar detail is used.

Details 19/4 and 20/4 Anchor and Gun Bolt Installation at Interior Non-Bearing Walls

The provisions shown on these details should be used at all interior non-bearing walls for slab on grade construction. When the walls are designed as shear walls, the appropriate detail may be used provided that strap- or angle-type hold-down anchors are not required.

Detail 21/4 Interior Shear Wall Connection to Wood Floor Framing

This detail is applicable when an interior shear wall is used and no continuous footing is provided beneath it. The shear must be transferred from the finish material to the bottom plate and thence to the floor sheathing. The blocking or double joist beneath the wall is supplied to provide proper backing for the nailing. The floor sheathing should be attached to the blocking or double joist with the same nailing required by Table 4.5 at the edges of sheets or at points of bearing.

Detail 22/4 Chord Splice

As stated in Chapter II—1, the top plates of exterior walls act not only as chords but also as struts carrying loads developed in the diaphragm to the shear-resisting elements along a given line of resistance. It is important, therefore, that proper nailing be provided at each splice in the plates. Although a minimum length between splices of 8′-0″ is indicated with a minimum length of 6′-0″ at the ends, the length between splices is not nearly as important as the provision for nine 16d nails minimum between each splice. The 8′-0″ minimum length indicated will allow the builder to do the least amount of nailing to achieve this requirement, since the nails between top plates are limited to 16″ maximum o.c. by the Nailing Schedule.

Detail 23/4 "Standard" Framing Anchor Use

Standardized framing anchors are produced by a number of manufacturers and come in a number of shapes, but generally follow the dimensions indicated. The BI and BO designations refer to whether the bendable portion is to be bent in or out. The method of installation and allowable seismic loads are indicated on the lower figures in the detail. Where framing anchors are indicated on other details, this type of framing anchor is intended. Smaller framing anchors are fabricated by some manufacturers and may be used to fasten 2 x 3 and 2 x 4 members in a manner similar to that shown in the lower left figure of Detail 23/4. When this smaller faming anchor is allowed, it is designated "2 x 4 framing anchor."

All "standard" framing anchors installed should have the minimum capacity for seismic resistance as shown on this detail. Where no seismic values for "standard" framing anchors are indicated by the manufacturer, the stated values for loads other than roof loads should be a minimum of 3/4 the values shown.

Detail 24/4 Fastening Studs to Sole Plates (Zone 3 only.)

The full detail is for use in Zone 3 only. The connection at the studs is applicable in Zone 2 also. In Zone 3, framing anchors are required not only at the exterior corners but at the base of the first two studs from each corner. In addition, framing anchors are required to be placed on the alternate sides of studs at 4'-0" o.c. at the first-floor exterior walls of two-story houses in Zone 3. In order to develop the transfer of load from the framing anchors to the anchor bolt, it is required that the sill plate be of a grade having a bending strength of 1,250 psi or better in both Zones.

Detail 25/4 Typical Strap Hold-Down (Cap. = 1,700 lbs)

Hold-down straps similar to the one detailed are commercially available in a variety of sizes and capacities. All straps indicated are minimum and may be fabricated from thicker material. *Straps such as the one shown in Detail 25/4 should be carefully installed; the most costly and troublesome problems occur when hold-downs are misplaced.* After concrete has been poured, engineering consultation is required to correct this type of error.

The double studs shown on Detail 25/4 should be placed such that the nails are approximately in the center of each stud; 4 x 4s or larger may be used in lieu of the double studs.

Detail 26/4 Framing Anchor Hold-Down (Cap. = 800 lbs max.)

This hold-down detail, similar to Detail 24, is to be used at both sides of all exterior corners. It is not used for commercial construction and has been devised exclusively for single-family residences. It will frequently be applied at locations other than corners since overturning in residential construction usually results in relatively low loads. Chapter II–6 includes a detailed discussion of all types of hold-down anchors.

Detail 27/4 Second Floor Strap Hold-Down (Cap. = 800 lbs max.)

Occasionally hold-down anchors are required at the ends of short shear walls occurring at the second-floor level. When a stud wall exists below the location of the hold-down strap, the strap should be fabricated long enough to attach the second-floor studs to matching first-floor studs. When a wall occurs over a beam or header, the attachment must obviously be made to this member.

Detail 27/4 is for a condition similar to Detail 26 as applied to first-floor construction. Since no anchor bolts can be installed, the strap is substituted for the framing anchors.

Detail 28/4 Typical Shear Wall Installation

Finish material should be applied as indicated in Detail 28/4, with the edge nailing of the adjoining sheets made to the same stud. The minimum allowable size of all sheet-type sheathing is 2'-0" x 4'-0". When narrower pieces could result from the normal installation of 4'-0"-wide sheets for the remainder of the length of the wall, the last 4'-0"-wide sheet should be cut back one stud space to allow the piece at the corner to be a minimum of 2'-0" wide. All splices of adjoining sheets should be made by nailing each piece to the same framing member (stud, top plate, blocking, etc.). Continuous shear-resisting material, such as plaster or stucco obviously need not meet the edge-nailing requirements stipulated above.

When angle-type hold-downs are used, it is usually necessary to supply a reinforced grade beam to resist the uplift transferrred to the foundation by the hold-down anchor. When any strap or angle hold-down is used, it is vitally important to nail the finish material to the same stud to which the hold-down is attached.

Details 29/4, 30/4 and 31/4 (including Table 4.2) Grade Beams at Shear Walls

These details are supplied primarily for the designer to determine the length of the reinforcing and grade beam required under a shear wall requiring angle-type hold-down anchors. The contractor may determine this distance in the field by noting the size and number of reinforcing bars required in the grade beam and referring to Table 4.2 (in Detail 31/4) to determine the "a" dimension. These details and the table apply to masonry as well as to wood stud shear walls.

Details 32/4 and 33/4 12" x 18" Grade Beams and 18" x 24" Grade Beams

When grade beams are required, reinforcing steel should be placed as shown on Details 32 and 33. Although no ties are indicated, the contractor may find that tying the bars is the simplest way to keep the reinforcing steel in place. When stem wall footings are used, the total cross sectional area of the footing and wall is to be the same as that shown for the more normal beam-shaped footings so that sufficient weight is present to resist the uplift from the hold-down anchor.

Detail 34/4 Installation of Angle Hold-Down Anchors

Hold-down anchors similar to the angle-type hold-down shown on Details 34A/4 through 34D/4 are commercially available from a number of manufacturers. If a plate washer is used under the nut on the vertical bolt, the hole in the angle can be fabricated to a diameter up to 1-1/8". This will allow greater tolerance in the placing of the hold-down bolt. It is recommended that the sill plate be cut and used as a template to better insure the proper placement of the hold-down bolt. Because of the cost of the hold-downs and the care with which they must be installed, it is fortunate that this type of anchor is rarely required in residential construction.

Provision must be made in advance for the application of finish material in order to provide coverage for the heads of the bolts on the opposite side of the double studs from the hold-down; 4 x 4 or larger posts may be substituted for the double studs. Hold-downs of this type occasionally are fabricated with plates attached at the bottom of the hold-down bolt rather than the hook shown. Such plates should be tack-welded to the embedded head of the bolt to prevent them from floating up when the concrete is poured. Detail 34A's dimensions apply to three other details as well.

Details 35/4 through 38/4 Miscellaneous Hold-Downs at Second Floor

The comments on Detail 27/4 apply to these details, where appropriate. Details 35 through 38 will rarely be used because they pertain to the installation of hold-down anchors on the second floor of two-story houses which are not commonly required.

It is emphasized that where a stud wall occurs below the hold-down anchor, matching double studs should be provided below the second-floor framing rather than attempting to attach the hold-down to a rim joist or other inadequate member. Headers to which hold-down anchors are fastened should be 4 x 8 minimum and, in turn, must be adequately anchored to transmit the uplift loads.

Details 39/4 through 44/4 Strut Connections

Struts as shown on these details indicate the required connections to the roof framing. To develop the strength of the strut, it is necessary that roof sheathing be nailed to the strut with minimum edge or bearing nailing required; the member must be doubled when indicated on the detail; and, primarily, the connection to the top plates of the wall must be as indicated. It is not necessary that the shear wall occur directly below the top plate at the connection. The shear-resisting portion of the line of resistance may be beyond the point at which the strut ties to the top plate. The top plates, themselves, will act as a strut to transmit the load to the shear-resisting element. As discussed in Parts I and II, very little damage is done to roofs during earthquakes; loads in residences seem to get to the walls with little difficulty. The development of struts in roof structures should be necessary only in isolated cases where most of the load for a given line of resistance is developed in a portion of the roof not in any way adjacent to the pertinent shear wall.

Although the struts shown indicate roof construction, the same connections may be utilized with second-floor framing. When the roof construction is shown as doubled to act as a strut, the floor framing should also be doubled. Struts in second floors will also occur infrequently, but should prove to be more common than those required at roof lines. Interior walls at the first floor of two-story homes are frequently staggered (in plan view) such that continuous plates cannot be provided to act as struts transferring the load from the floor diaphragm to the shear wall.

The contractor should be aware of the need for struts and, to provide himself with a guarantee of economy, should question any strut connected to a wall with shear-resisting material having a shear value of less than 250 pounds per foot. On occasion, struts will be justified but, generally speaking, the length of the diaphragm adjacent to the shear wall should be sufficient for load transference without a strut when the shear-resisting material has a capacity less than the 250 pounds.

Struts parallel to framing are easily installed because the framing itself can be used as the strut. The cost of installation is primarily the cost of the connection and the time required to provide the few extra nails from the roof or floor sheathing to the strut. When the strut is perpendicular to the roof or floor framing, the connection becomes more complicated and costly because straps must be used and blocking must be installed between the rafters or floor joists. The vertical strap indicated on Details 42/4 and 44/4 is required only when the roof slopes. When the roof (or floor) is level, or nearly level, this vertical strap may be omitted. Struts of the sort detailed develop loads in two ways: When the strut is in compression, the force is translated directly to the blocking or rim joists at the point where the members abut. When the strut acts in tension, however, the strap or framing anchor must transfer the load to the top plates of the wall. Straps in particular are inadequate in compression because they tend to buckle, but are the only means of transfer for tension forces. It is important, therefore, that straps be installed as tightly as possible to minimize the tendency to buckle when the strut is in compression.

Detail 45/4 Special Garage Front Wall Detail – One-Story Garages

This detail is provided for use at the front of one- and two-car garages and should be used at the wall on each side of the garage door when each wall is less than 4'-0" in width.

Detail 45/4 should be used for detached or attached garages whenever the garage is not in line with other exterior walls of the house or whenever the horizontal distance between an exterior wall of the house and the front of the garage is greater than 6'-0".

Rigid frames constructed of wood are normally considered questionable engineering practice. Detail 45/4 has been developed as a compromise between more expensive solutions and the current inadequate performance at the front of garages. The frame is developed through use of a larger-than-normal header strapped to 4 x 4 posts. The header extends across the full width of the garage rather than stopping at the trimmer at the edge of the opening. This method of attachment precludes the necessity of installing hold-down anchors, which are impractical, if not impossible, to install in the shorter segments of wall allowed. The plywood and the straps may be placed on either the interior or exterior face of the wall. It is not necessary that the straps be placed on the same side of the wall as the plywood but, when that is the case, the straps should be installed over the plywood. To preclude the necessity of other filler behind the portions of the garage wall not requiring the plywood sheathing, it usually will be more desirable to place the plywood on the interior of the wall as shown on the upper plan on Detail 45C.

When three-car garages are constructed, a "pier" should occur either between the double door and the single door or between each of the single doors. At least one of these interior "piers" (as well as the end "piers") should be developed in a manner similar to that shown except that the header and straps must be treated differently at the center "pier." One of the headers should be run across the full width of the center pier with the other header supported on a trimmer placed adjacent to the 4 x 4 posts. The plywood should be nailed to both posts as shown (but not necessarily to the trimmer) and the five-bolt strap should be installed at the side of the pier farthest away from the end of the header that extends across the pier.

Detail 45/4 should not be used with walls 4'-0" or greater in width because such walls may be considered as shear walls and can be developed in a more conventional manner.

Detail 46/4 Special Garage Front Wall Detail – Two-Story Construction

The same restrictions on the use of Detail 45/4 apply to Detail 46/4. Plywood sheathing is necessary on both sides of the wall in Detail 46/4, however, to carry the heavier loads generated in two-story construction. U straps and 4 x 6 posts also are required.

Inasmuch as plywood must be applied to both sides of the wall, it will be necessary to provide filler material over the remainder of the wall in order to prevent a "hump" in the finish material at the location of the plywood.

Details 47/4 and 48/4 Typical Fireplace Construction

Masonry fireplaces must be reinforced and tied to the roof and floor framing in accordance with Detail 47/4.

Detail 48/4 indicates that nails may be used in the strap connecting the chimney to the second floor. Nails were found to perform poorly in conjunction with fireplace straps during the San Fernando earthquake. Although the number of nails shown by this detail is substantially greater than were placed in strap ties at the time of the San Fernando quake, the installation of 1/2" round bolts in each block is encouraged in lieu of the nails shown.

Metal fireplace ties are covered by the Design Methodology in Part III of this report.

Detail 49/4 Typical Water Heater Installation

The overturning of gas water heaters with the resultant break in connections can result in destruction by fire of an otherwise undamaged home. It is easy to understand how water heaters overturned when flexible couplings were used. More surprisingly, rigid piping did not prevent overturning of the water heaters in the San Fernando earthquake and therefore the connection shown in Detail 49/4 is required in both instances. The 16-gage angle indicated for the connection is not commercially available as a prefabricated item. Until it is, it will be necessary for contractors to have similar angles fabricated to the length required for each specific job. Although it is not necessary from a structural standpoint, it may be simpler to have the angles made up from standard structural steel rather than with light gage steel.

Detail 49/4 emphasizes the necessity to tie water heaters near their tops and to provide mechanical fastenings near the bases to prevent sliding as well as to assist in preventing buckling of the water heater legs. It is recognized that this detail will not meet all water heater configurations, or that an individual contractor may wish to provide a different installation. This is acceptable as long as the connections provide resistance to overturning in any direction, and as long as a positive mechanical fastening is provided at the base to assist in the prevention of sliding.

Detail 50/4 Cabinet Fastenings

Detail 50/4's provisions for the installation of cabinets and other similar items are not unlike those currently in use. The major difference is the requirement of screws rather than nails as fasteners. Some installers already use this method and it should not greatly affect the cost of installation.

Details 51/4 and 52/4 Masonry Wall Reinforcing

These wall elevations and their accompanying cross sections are essentially the same, with fully-reinforced masonry walls indicated on Detail 51/4 and partially-reinforced walls (applicable to Zone 2 only) shown on Detail 52/4. Horizontal joint reinforcing may be used in lieu of the horizontal reinforcing bars as shown in one elevation on each sheet. It is emphasized that the masonry lintel reinforcing indicated is the minimum required for seismic load and it is not implied that this reinforcing is adequate for the vertical loads in any individual home. When the lintel span is short it is not uncommon to place the channel block as indicated in the upper left hand section of Detail 51C or 52C. For longer spans this block should be inverted to provide a greater depth for the reinforcing steel.

Details 53/4 and 54/4 Anchorage of Bearing and Non-Bearing Masonry Walls

These details are intended to convey the same information indicated on Details 1/4 and 2/4 for wood frame construction. Because of the heaviness of masonry walls and the change in materials it is necessary that anchor bolts be used to connect the wood to the masonry for shear transfer. These bolts should be sized and spaced as indicated on the details. In addition, in Zone 3 only, it is required that joist anchors be installed in order to support the wall for loads perpendicular to it. In Zone 2, a lighter connection is acceptable – Details 53B and 54B.

Detail 55/4 Anchorage of End Walls at Gable Roofs

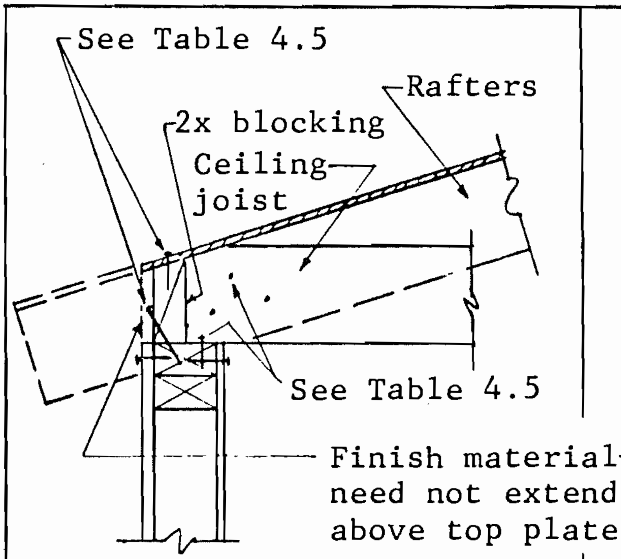
Two alternative details are presented to achieve the anchorage of walls stopping at the ceiling level with end trusses above. The detail for Zone 3 is considerably heavier than the detail shown for Zone 2. Detail 55A is a more positive connection since it connects directly into the roof itself. When the ceiling material is applied using the nailing schedule shown in Table 4.5, the wall may also be anchored into the ceiling as shown on Detail 55B.

Details 56/4 and 57/4 Connection of Masonry Wall at Second Floor of Two-Story Home

Masonry wall construction is limited in this publication to one-story heights. Higher structures should be engineered. When the masonry wall is extended to the second-floor line and wood stud walls are constructed above, Details 56/4 and 57/4 apply depending upon the direction of the floor joists.

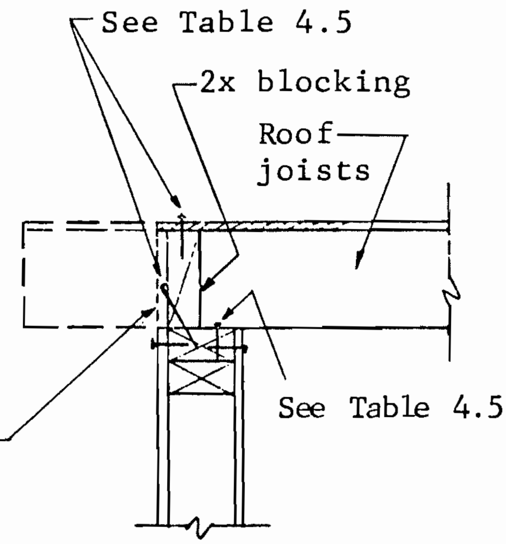
Details 58/4 through 60/4 Footings at Exterior Masonry Walls

In normal masonry construction it is unnecessary to anchor the first floor to the wall for loads perpendicular to the wall. It is necessary, however, to provide anchor bolts to effect a shear transfer between the first-floor diaphragm and the wall. When the masonry wall exceeds 4′-0″ in height below the floor line, consideration should be given to installing joist anchors in Zone 3 similar to the detail shown at the roof. At slabs on grade Detail 60/4 is presented simply to indicate the laps required in the reinforcing steel. The construction of the footing and slab itself should be in accordance with Detail 15/4.



DETAIL 1A/4

STANDARD ROOF FRAMING



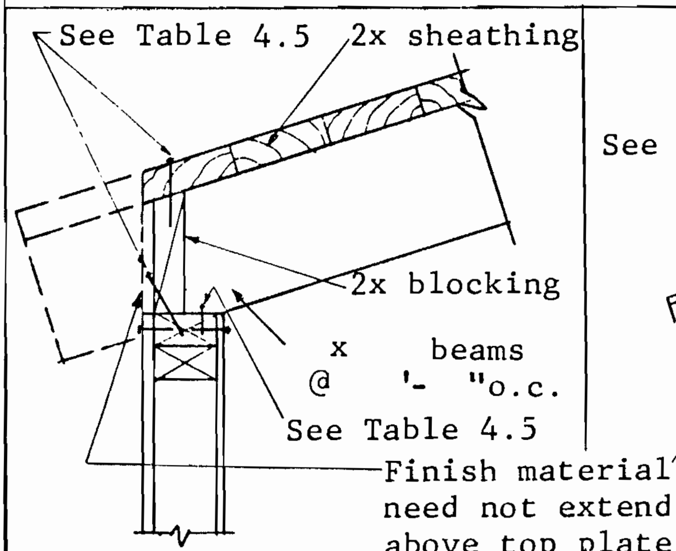
DETAIL 1B/4

FLAT ROOF FRAMING

EXTERIOR BEARING WALLS AT ROOF

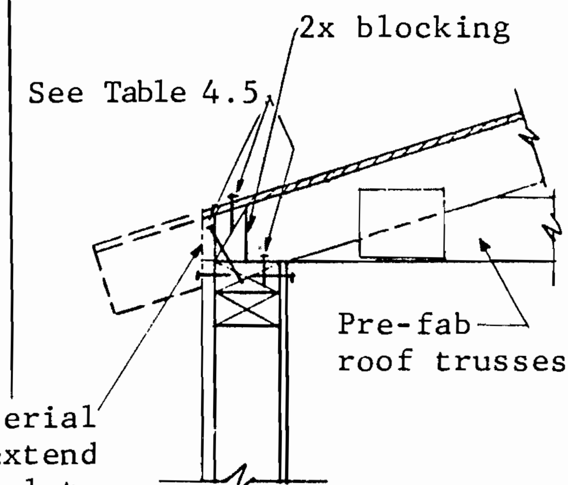
Notes: 1. Finish materials to be fastened per minimum requirements of Table 4.3, except as otherwise noted on plans.

2. Fasten exterior finish material directly to blocking where no roof overhang occurs.



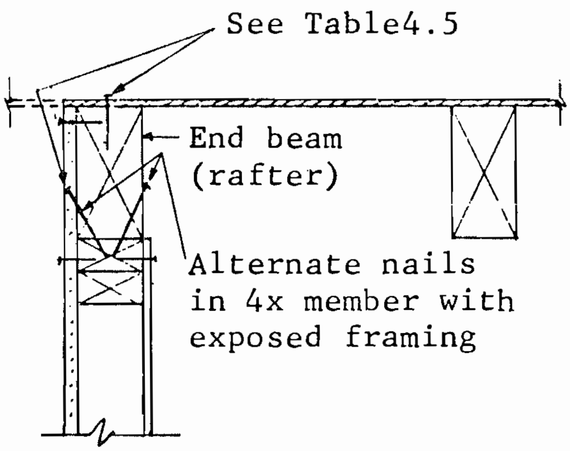
DETAIL 1C/4

EXPOSED BEAM ROOF FRAMING



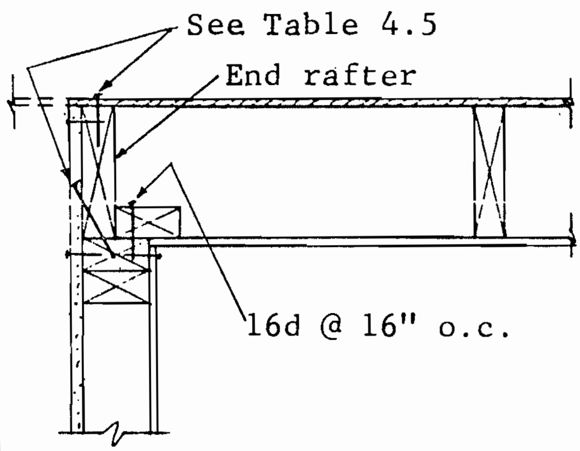
DETAIL 1D/4

PRE-FAB TRUSS ROOF FRAMING



DETAIL 2A/4

EXPOSED BEAM ROOF FRAMING



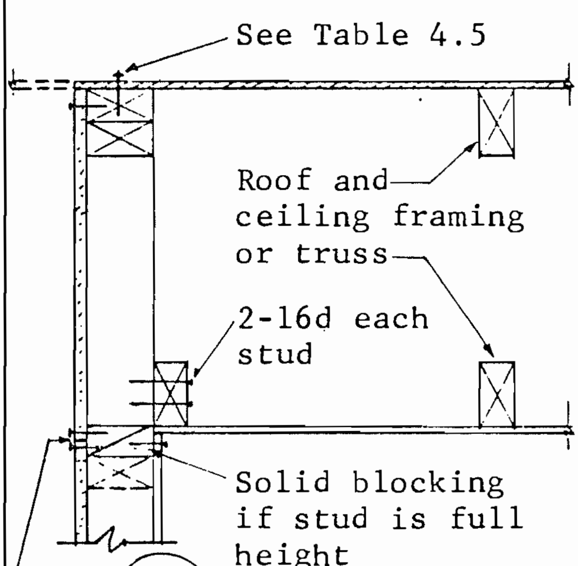
DETAIL 2B/4

FLAT ROOF FRAMING

EXTERIOR END WALLS AT ROOF

Notes: 1. Finish materials to be fastened per minimum requirements of Table 4.3, except as otherwise noted on plans.

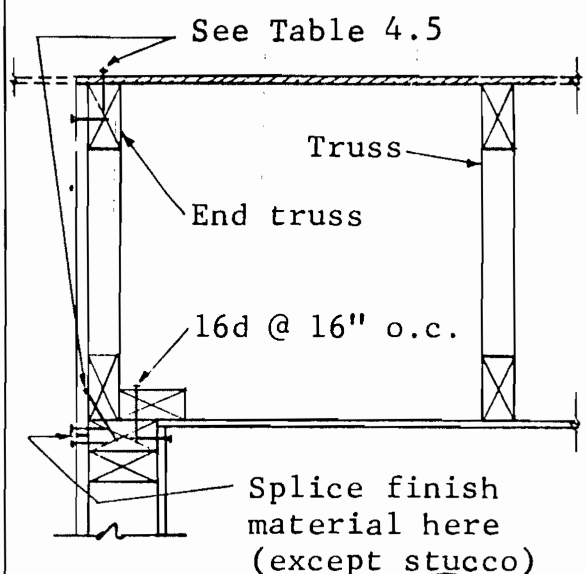
2. Exterior finish material to be fastened to end rafter, end truss or top plate.



See 2D/4

DETAIL 2C/4

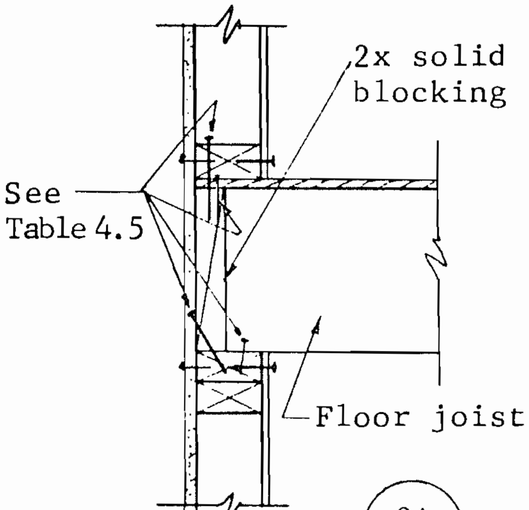
STANDARD OR TRUSS FRAMING



DETAIL 2D/4

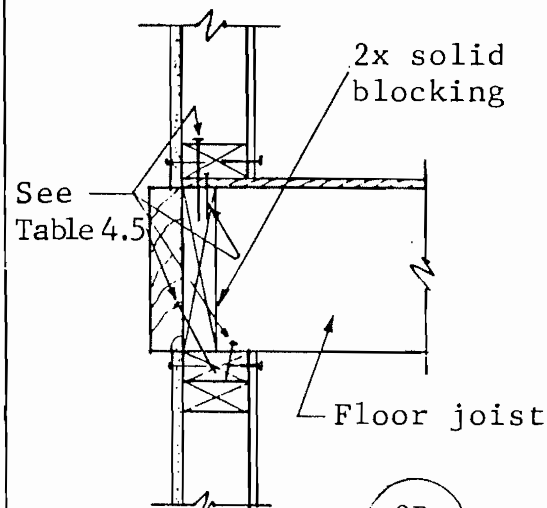
TRUSS FRAMING

EXTERIOR FINISH CONTINUOUS
PAST FLOOR FRAMING



DETAIL 3A/4

EXTERIOR FINISH INTERRUPTED
AT FLOOR FRAMING

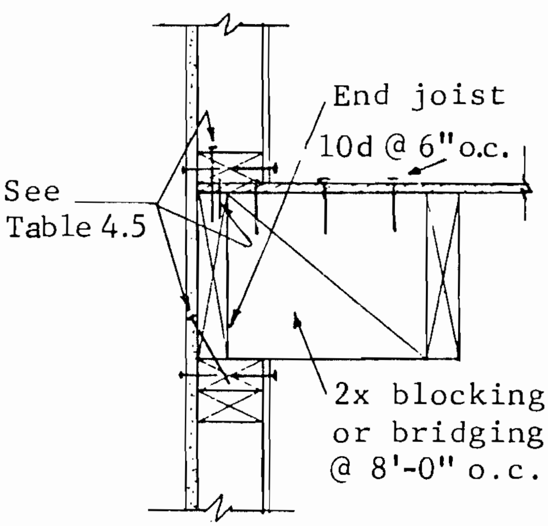


DETAIL 3B/4

EXTERIOR WALLS AT SECOND FLOOR

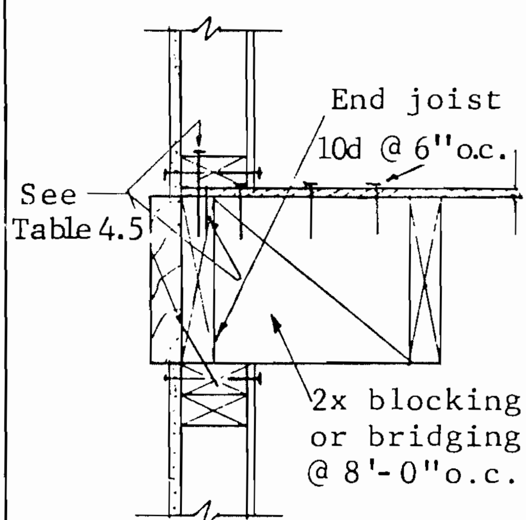
Notes: 1. **Finish materials to be fastened per minimum requirements of Table 4.3, except as otherwise noted on plans.**

2. **Where finish material is continuous past blocking, structural attachment shall also be continuous.**



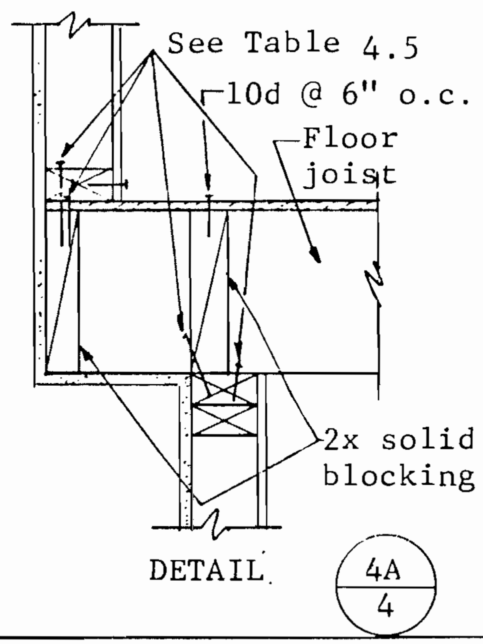
DETAIL 3C/4

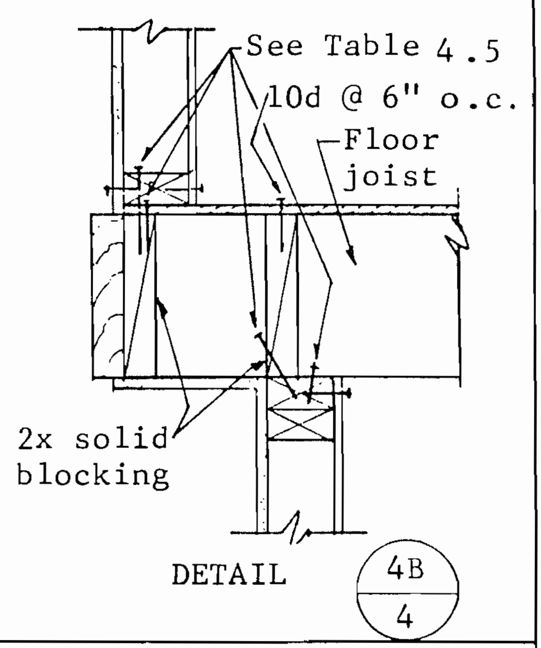
EXTERIOR FINISH CONTINUOUS
PAST FLOOR FRAMING



DETAIL 3D/4

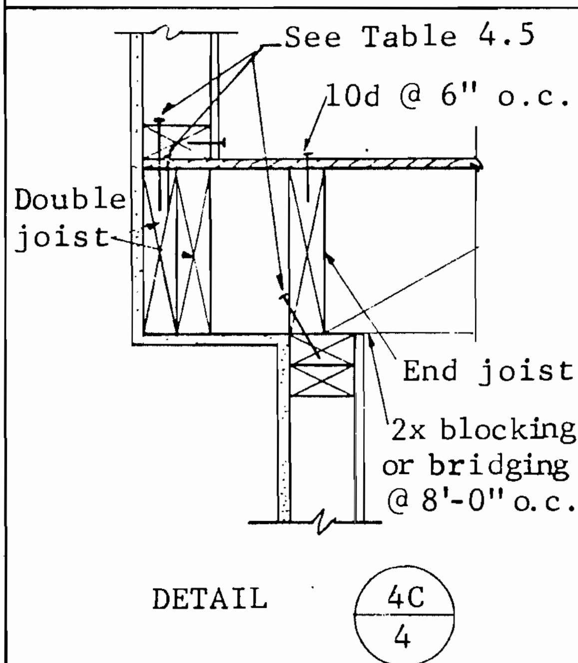
EXTERIOR FINISH INTERRUPTED
AT FLOOR FRAMING

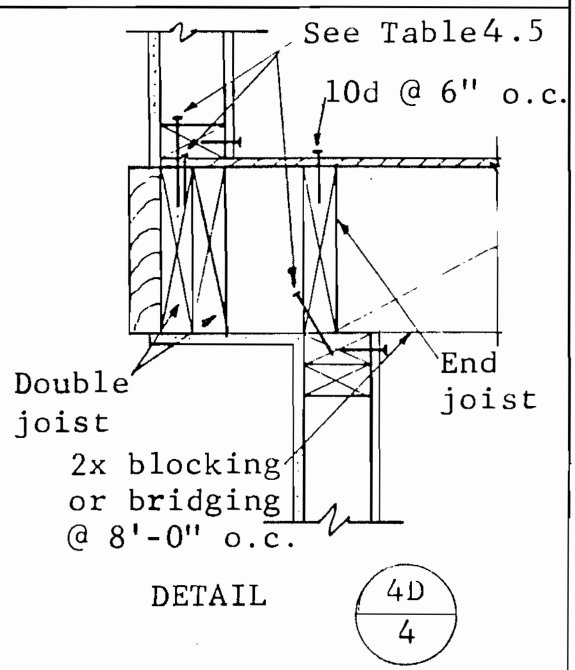


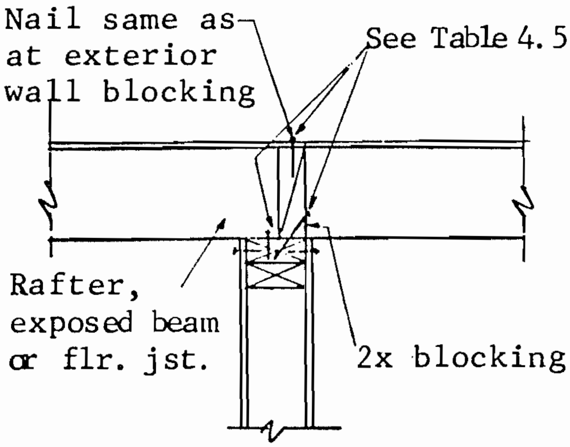


EXTERIOR WALLS – SECOND FLOOR OVERHANG

Note: 1. Finish materials to be fastened per minimum requirements of Table 4.3, except as otherwise noted on plans.

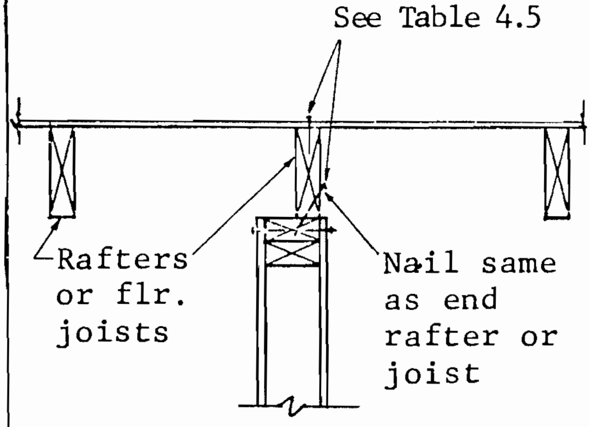






DETAIL 5A/4

INTERIOR BEARING WALL



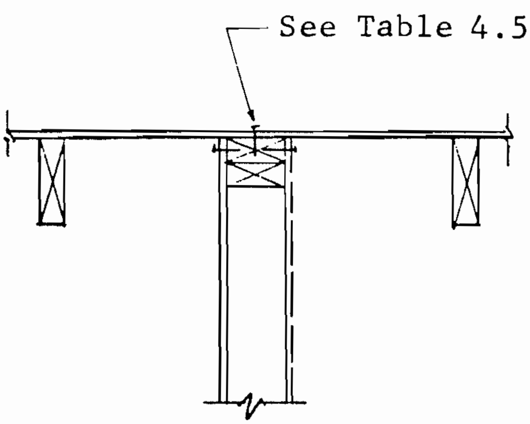
DETAIL 5B/4

INTERIOR NON-BEARING WALL

INTERIOR SHEAR WALLS

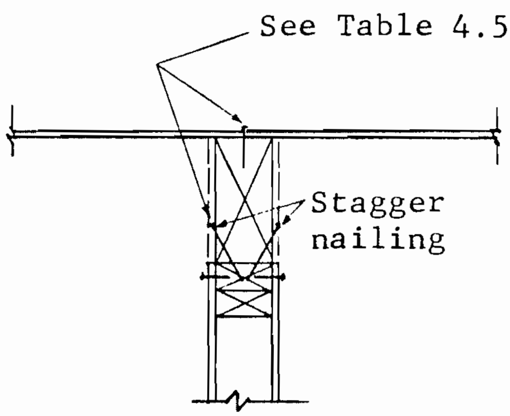
Notes: 1. Finish materials to be fastened per minimum requirements of Table 4.3, except as otherwise noted on plans.

2. Fasten finish material directly to blocking or framing member when finish extends to roof sheathing.



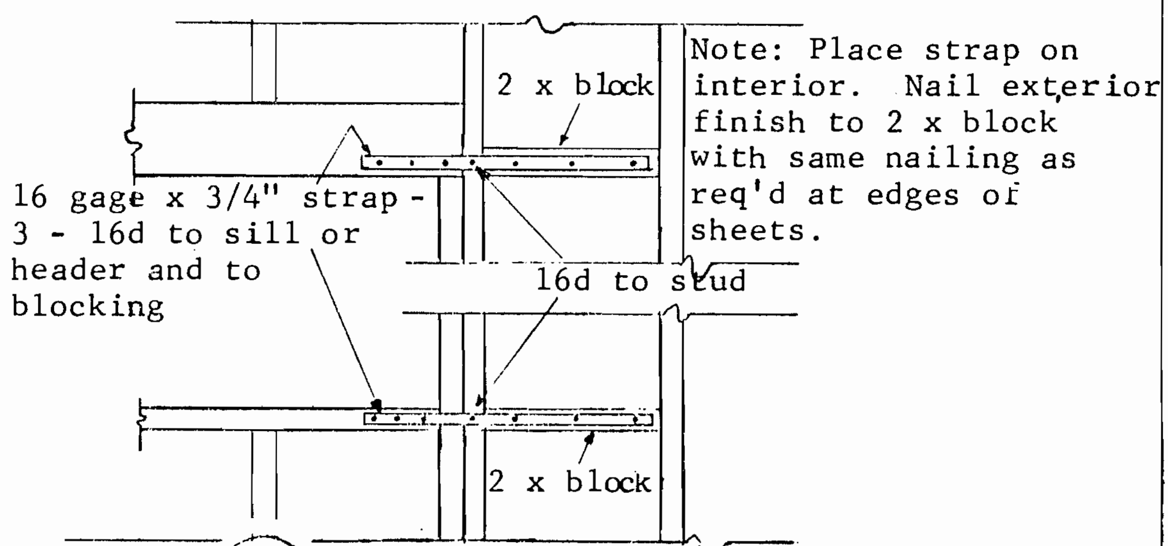
DETAIL 5C/4

INTERIOR NON-BEARING WALL
ALTERNATE DETAIL



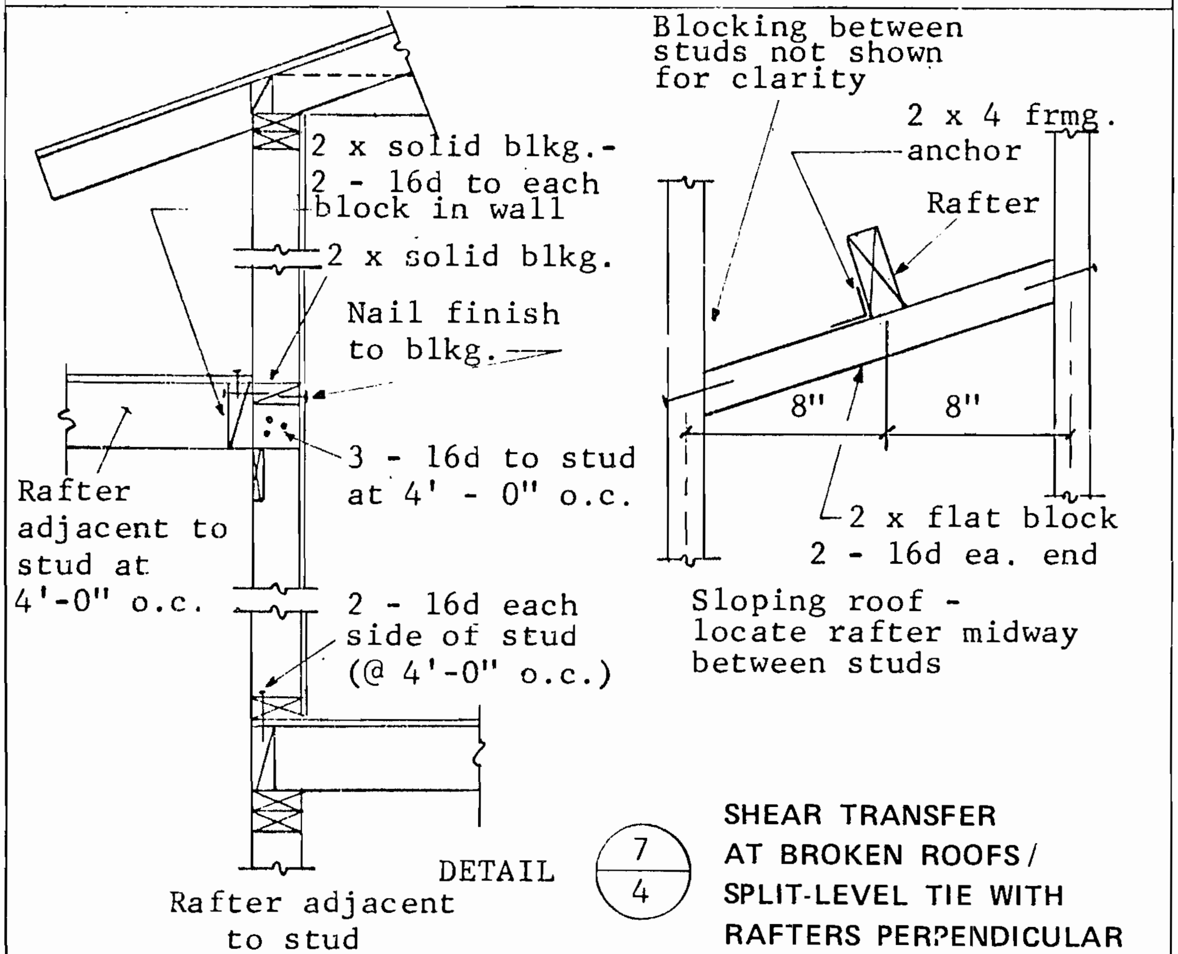
DETAIL 5D/4

INTERIOR WALL WITH
EXPOSED BEAMS



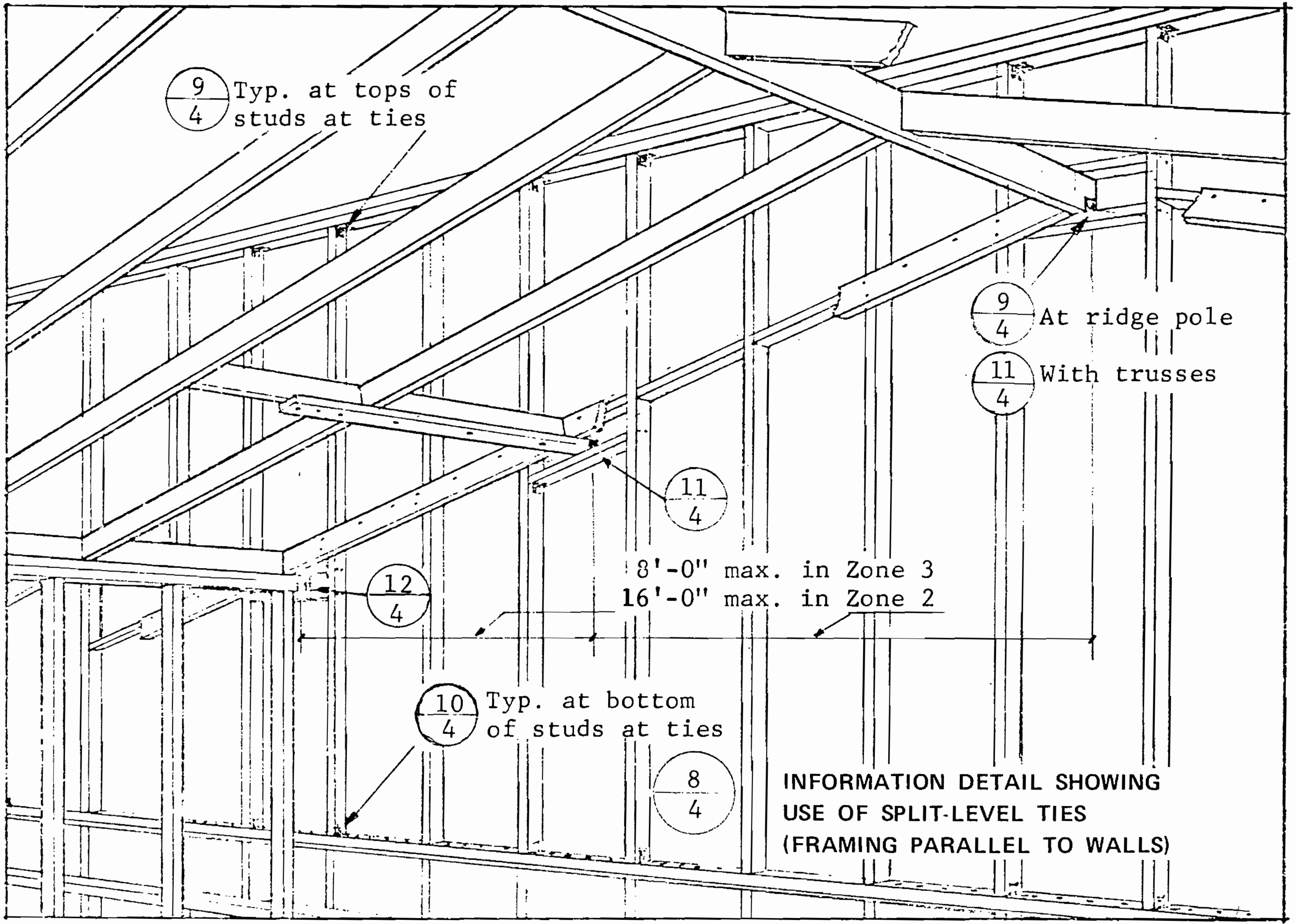
OPTIONAL DETAIL 6/4

BRACING AT CORNERS OF OPENINGS

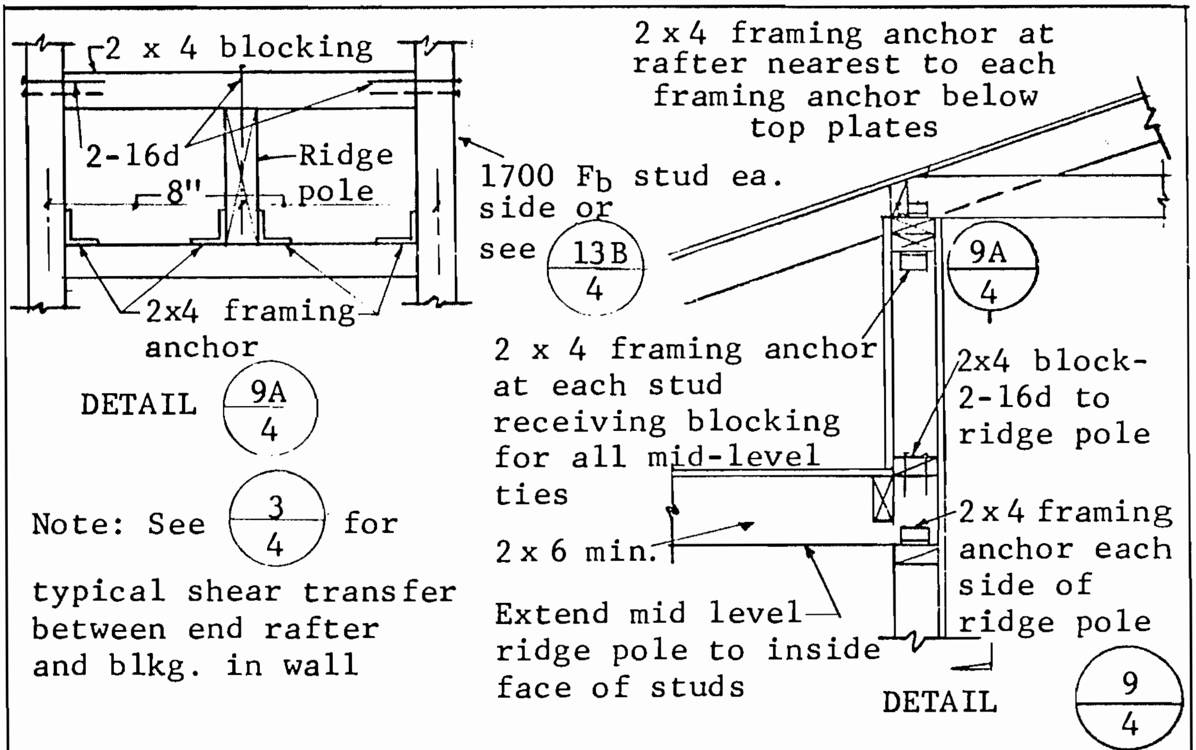


DETAIL 7/4

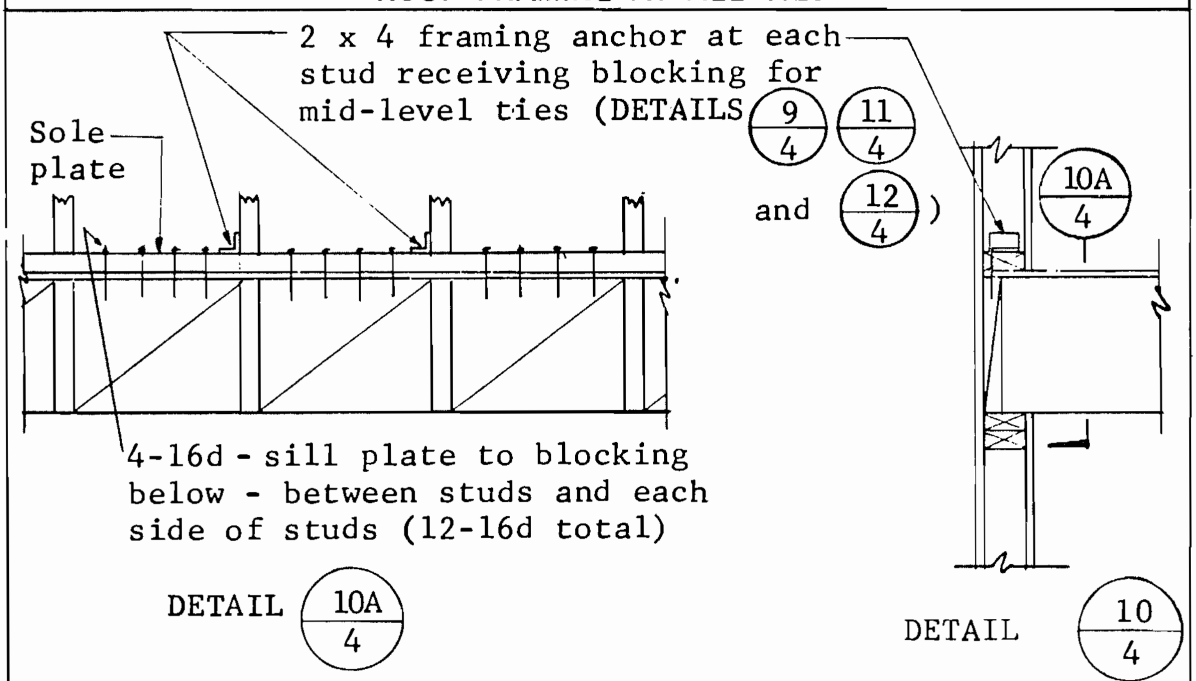
SHEAR TRANSFER AT BROKEN ROOFS / SPLIT-LEVEL TIE WITH RAFTERS PERPENDICULAR



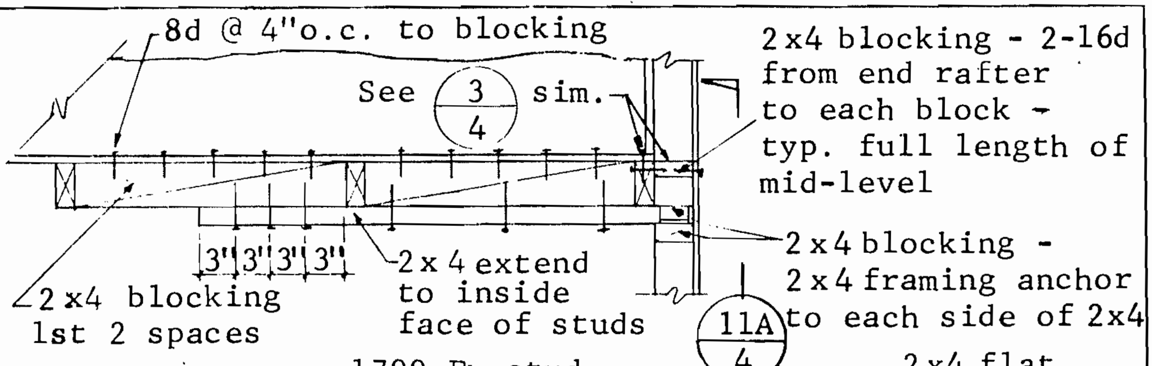
INFORMATION DETAIL SHOWING
USE OF SPLIT-LEVEL TIES
(FRAMING PARALLEL TO WALLS)



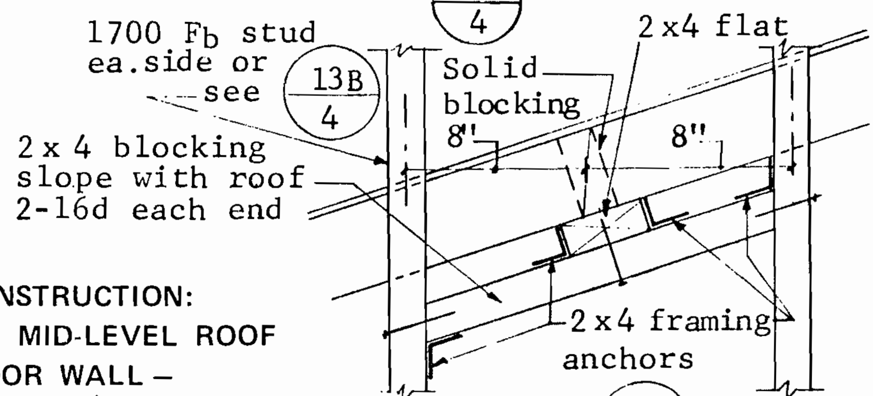
SPLIT-LEVEL CONSTRUCTION: CONNECTION OF MID-LEVEL ROOF TO SECOND FLOOR WALL – RIDGE TIE AND CONNECTION OF STUDS TO ROOF FRAMING AT ALL TIES



SPLIT-LEVEL CONSTRUCTION: CONNECTION OF STUDS TO FLOOR FRAMING AT ALL TIES

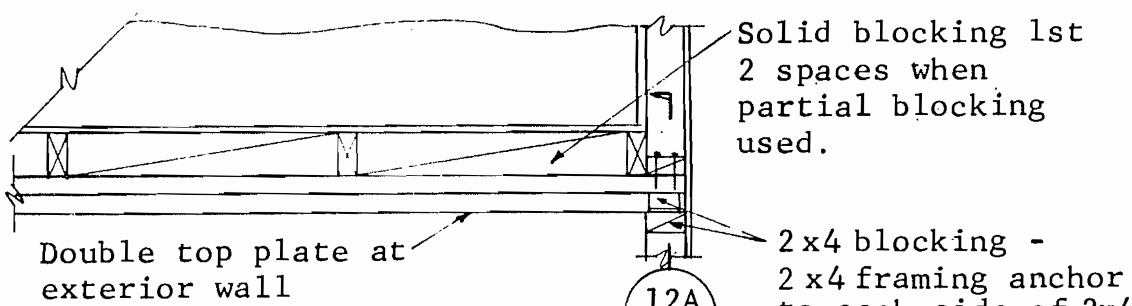


DETAIL 11/4



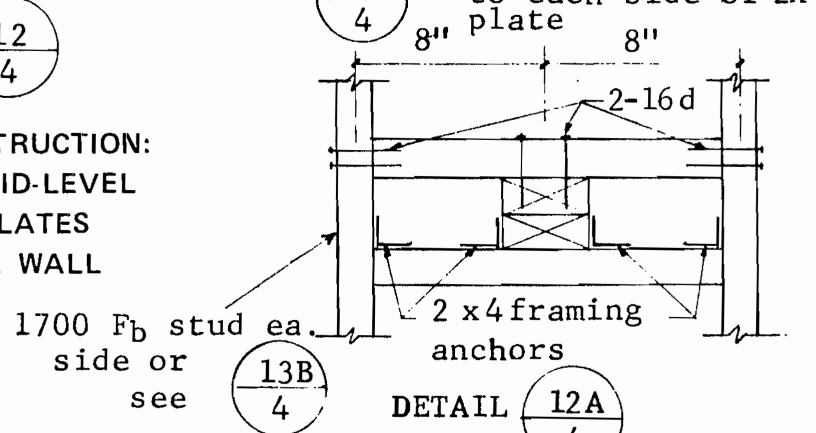
DETAIL 11A/4

SPLIT-LEVEL CONSTRUCTION: CONNECTION OF MID-LEVEL ROOF TO SECOND FLOOR WALL – INTERMEDIATE TIES (USE ALSO AS RIDGE TIE WITH TRUSSES)

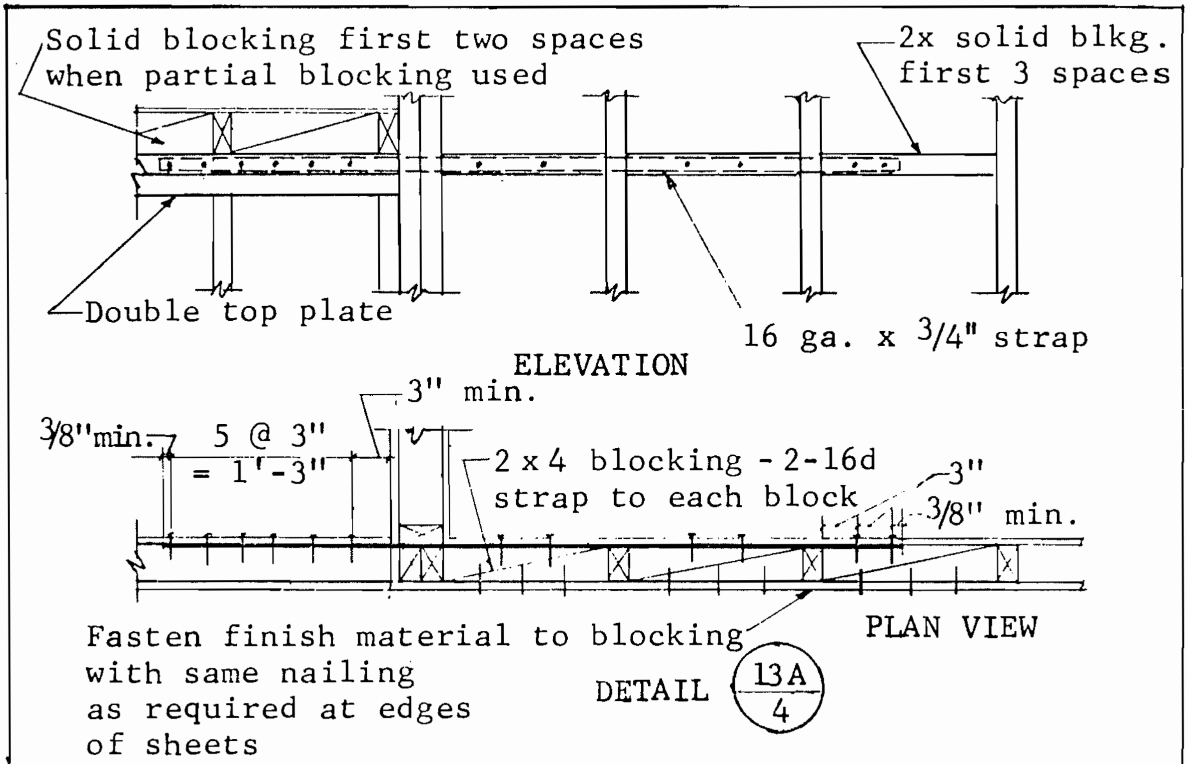


DETAIL 12/4

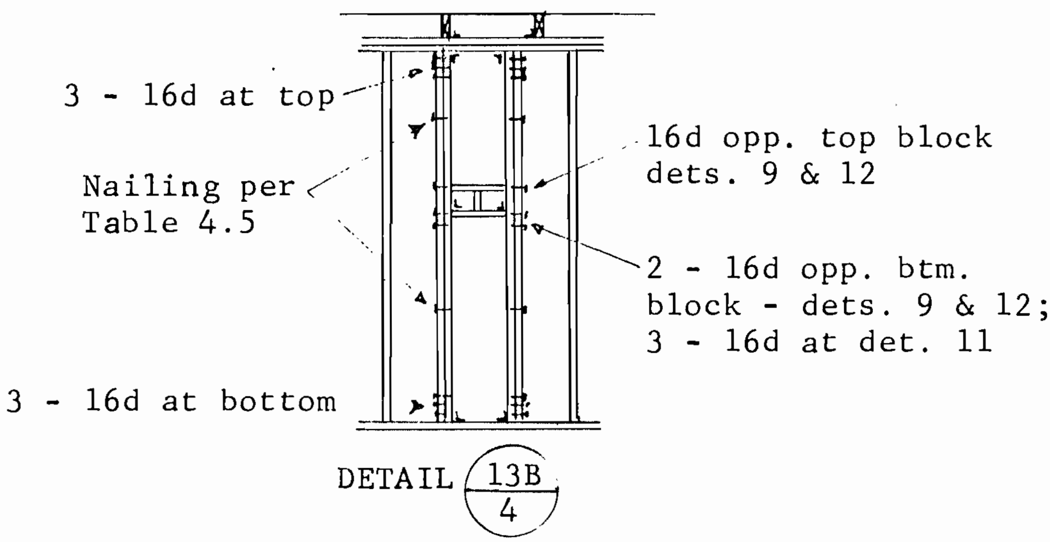
SPLIT-LEVEL CONSTRUCTION: CONNECTION OF MID-LEVEL EXTERIOR WALL PLATES TO SECOND FLOOR WALL



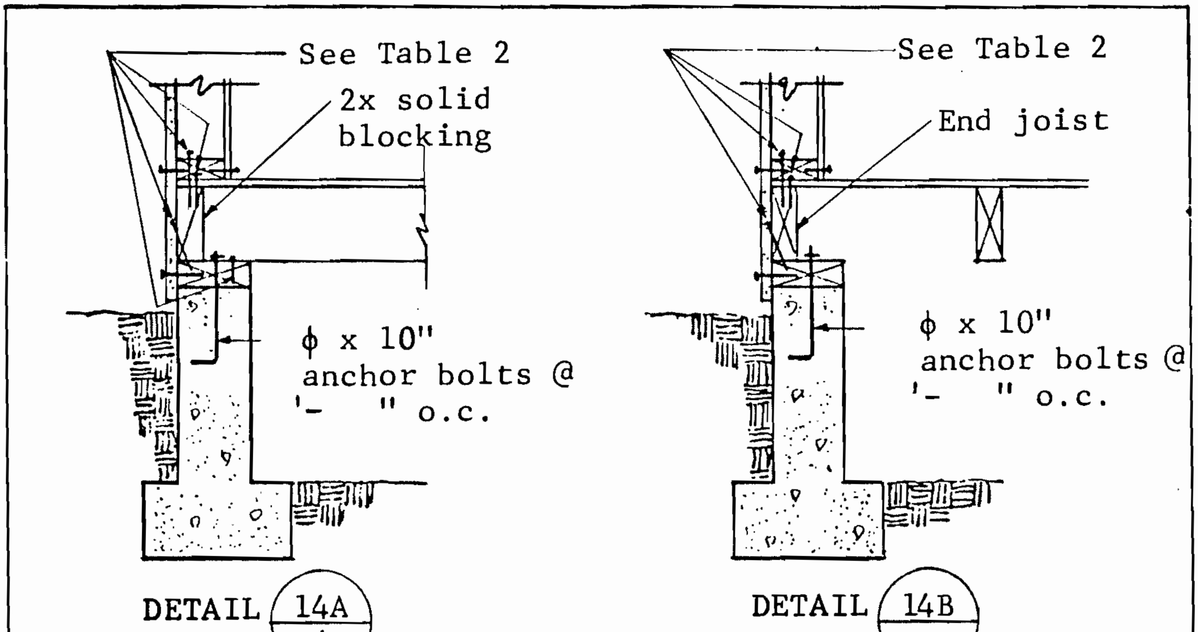
DETAIL 12A/4



SPECIAL DETAIL FOR SPLIT-LEVEL CONSTRUCTION: CONNECTION OF MID-LEVEL EXTERIOR WALL PLATES TO TWO-STORY WALL

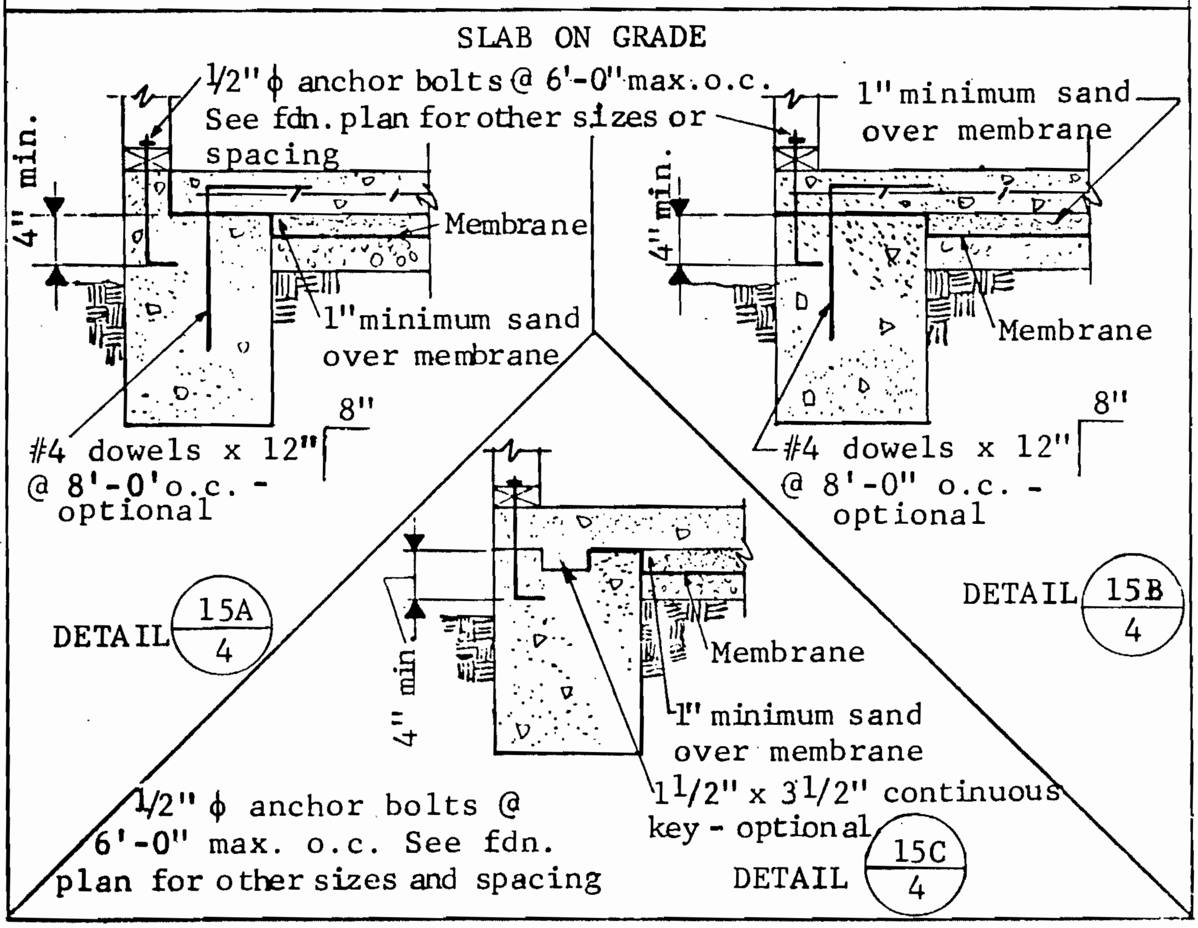


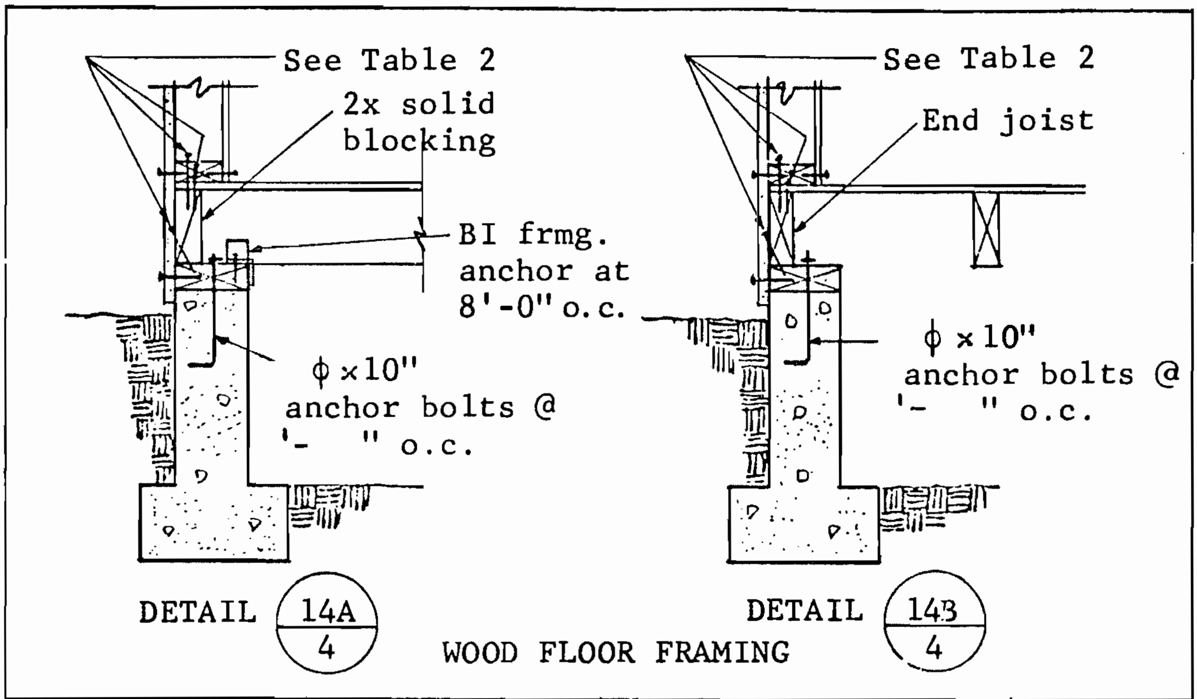
SPECIAL DETAIL: DOUBLE STUDS EACH SIDE OF SPLIT-LEVEL TIE IN LIEU OF USING 1700 F_b STUDS



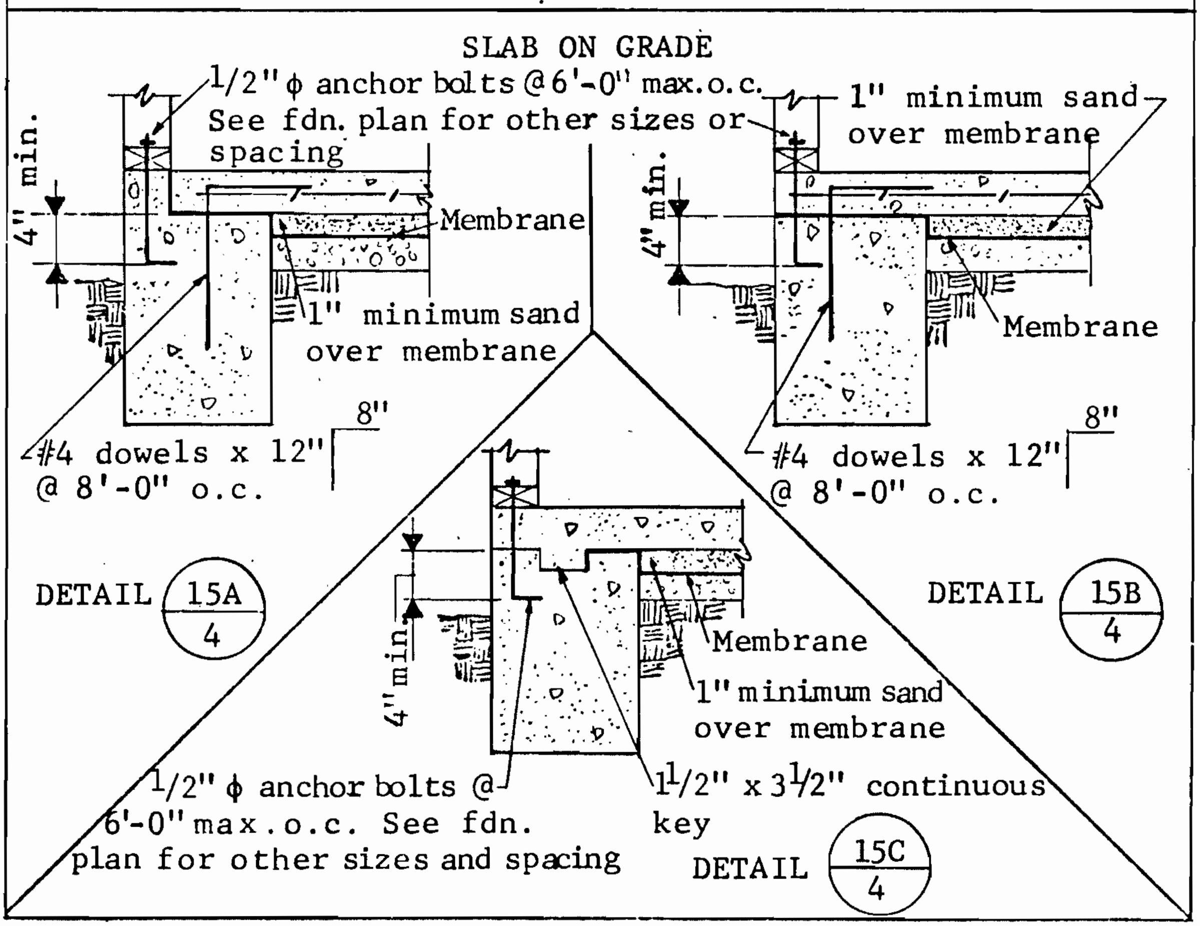
WOOD FLOOR FRAMING

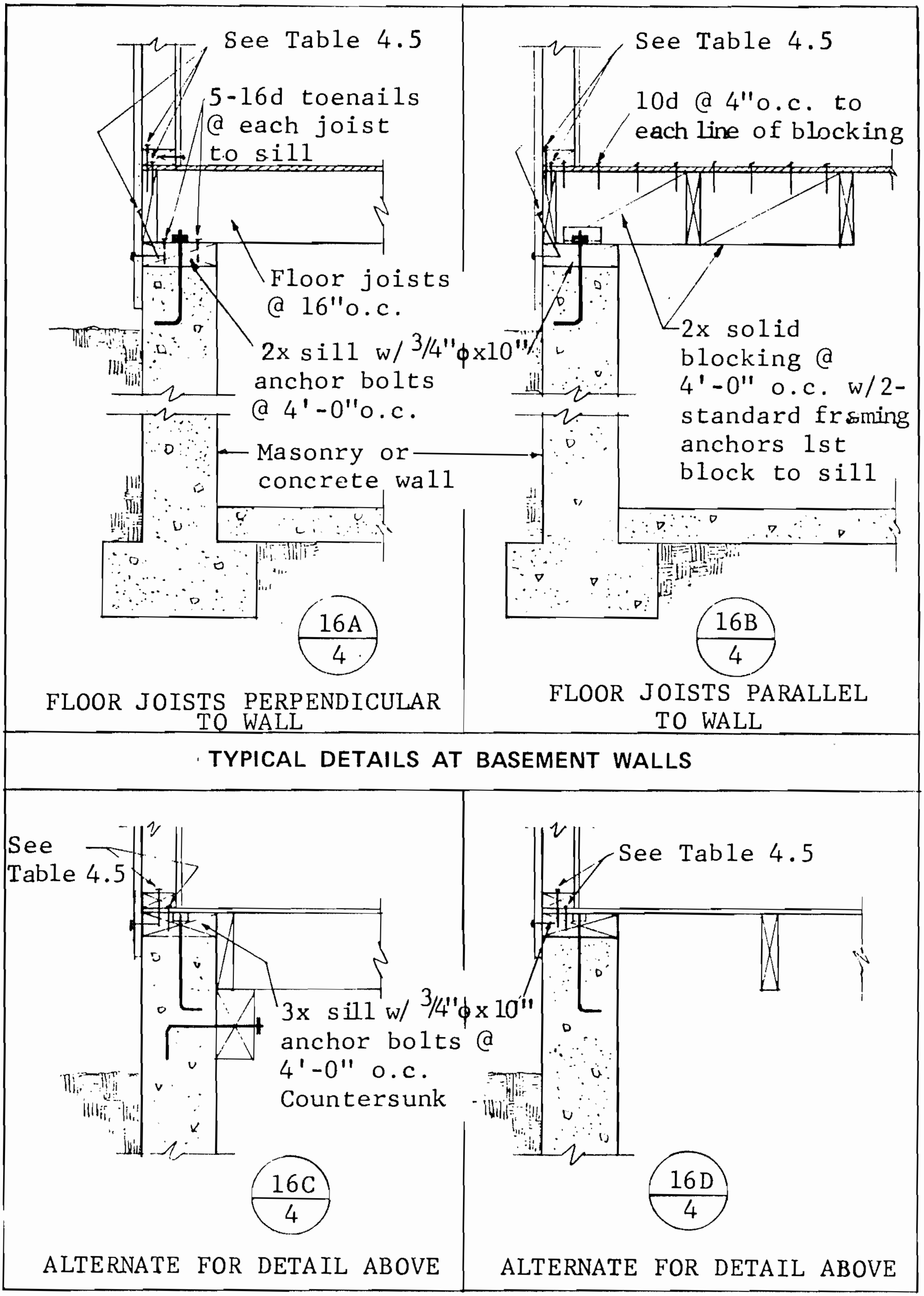
TYPICAL FOOTING DETAILS – ZONE 2





TYPICAL FOOTING DETAILS – ZONE 3





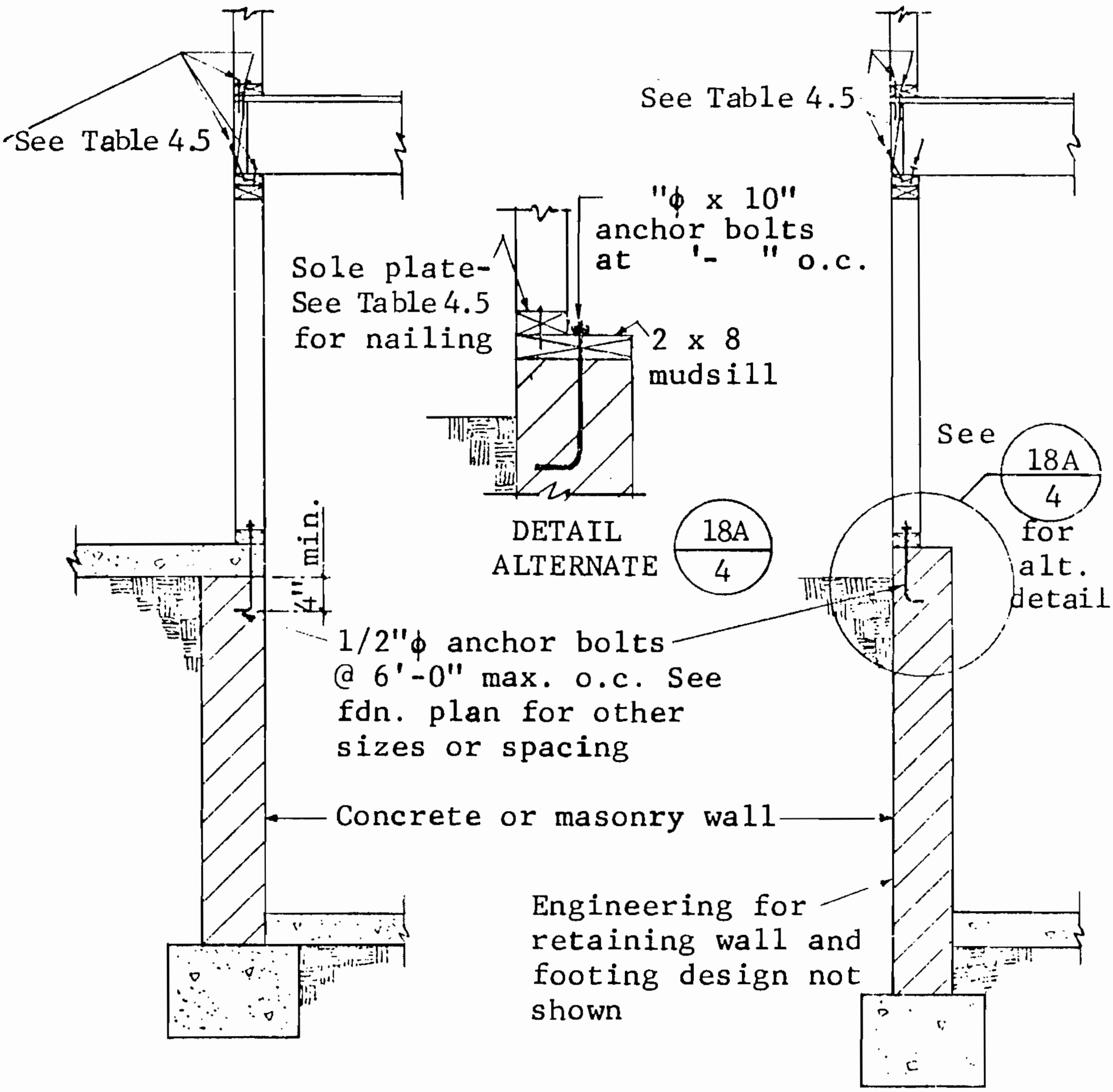
FLOOR JOISTS PERPENDICULAR TO WALL

FLOOR JOISTS PARALLEL TO WALL

TYPICAL DETAILS AT BASEMENT WALLS

ALTERNATE FOR DETAIL ABOVE

ALTERNATE FOR DETAIL ABOVE

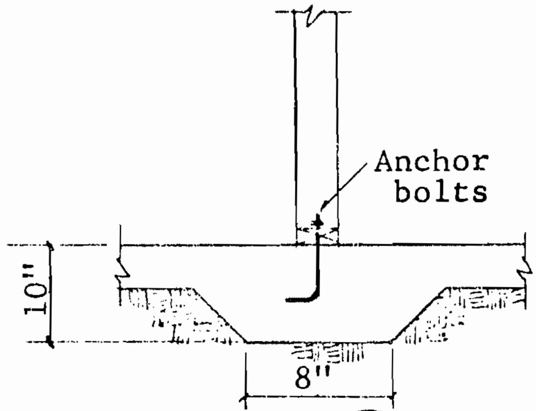


DETAIL 17/4

RETAINING WALL
AT SPLIT-LEVEL
CONSTRUCTION

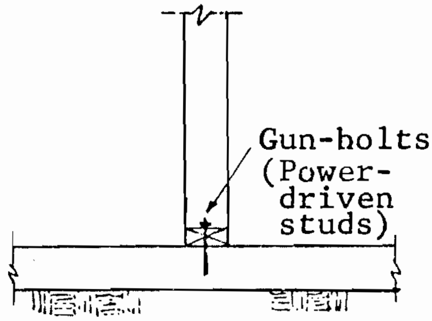
DETAIL 18/4

RETAINING WALL
AT KNEE WALL
CONSTRUCTION



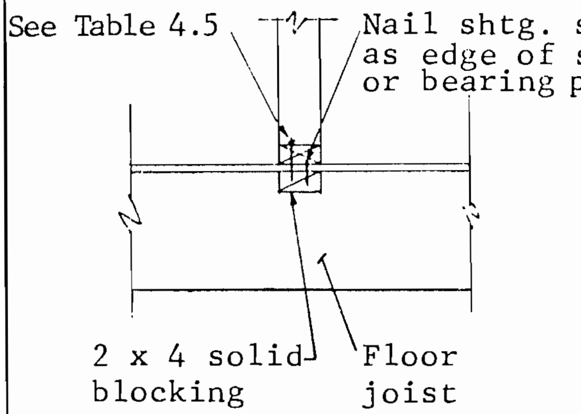
DETAIL 19/4

INSTALLATION
OF ANCHOR BOLTS
AT INTERIOR NON-
BEARING WALLS
(WHEN NO HOLD-DOWN
IS REQUIRED)



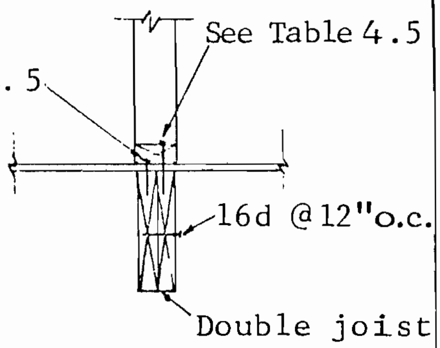
DETAIL 20/4

INSTALLATION
OF GUNBOLTS
AT INTERIOR NON-
BEARING WALLS
(WHEN NO HOLD-DOWN
IS REQUIRED)



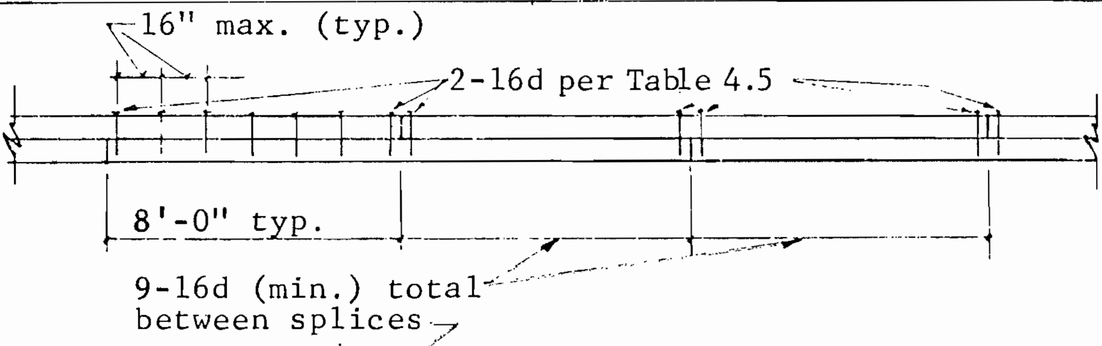
DETAIL 21A/4

INTERIOR SHEAR WALL CONNECTION TO PERPENDICULAR WOOD FLOOR FRAMING

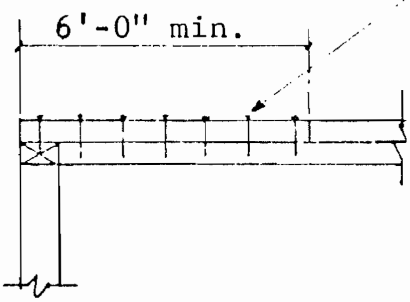


DETAIL 21B/4

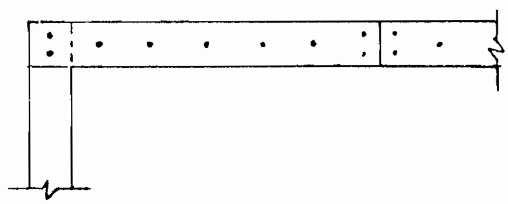
INTERIOR SHEAR WALL CONNECTION TO PARALLEL WOOD FLOOR FRAMING



TYPICAL TOP PLATE SPLICE



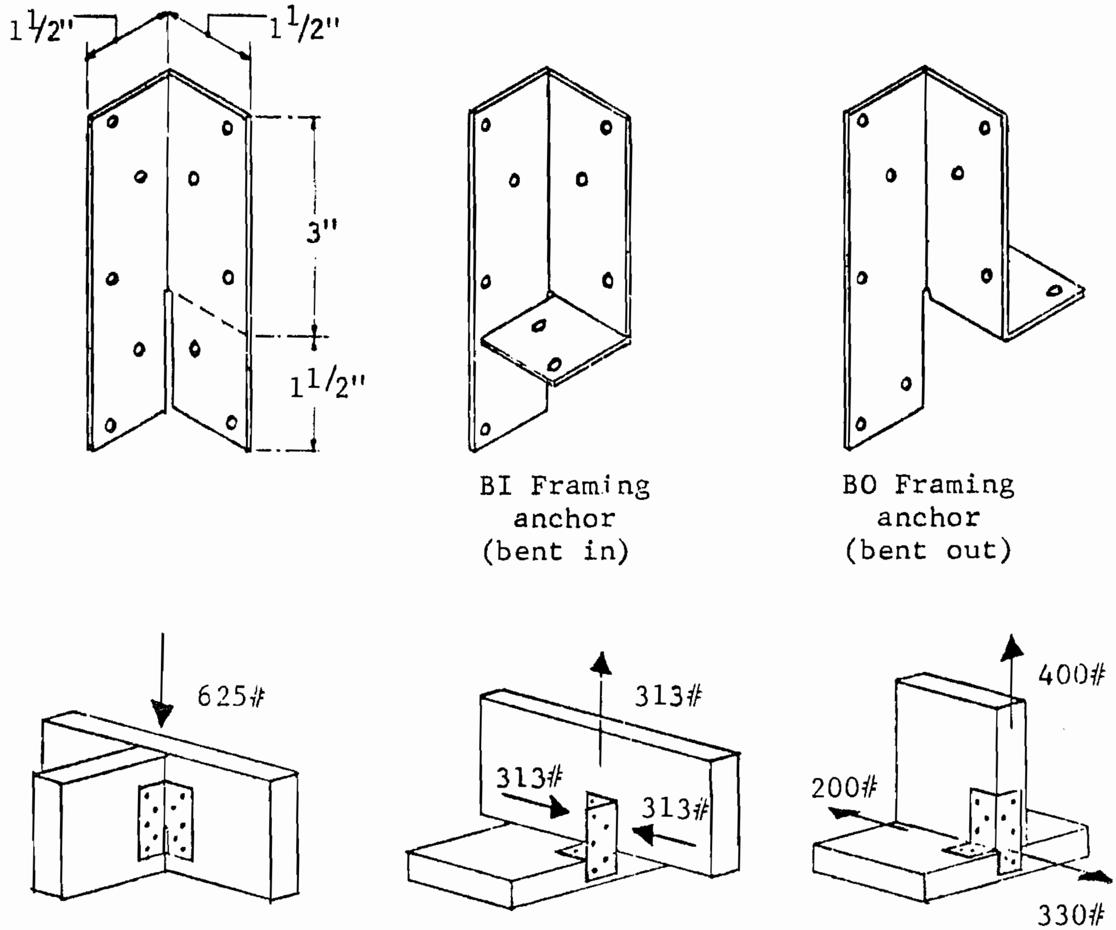
ELEVATION



PLAN

TYPICAL SPLICE AT EXTERIOR CORNER

DETAIL 22/4 TYPICAL CHORD SPLICES



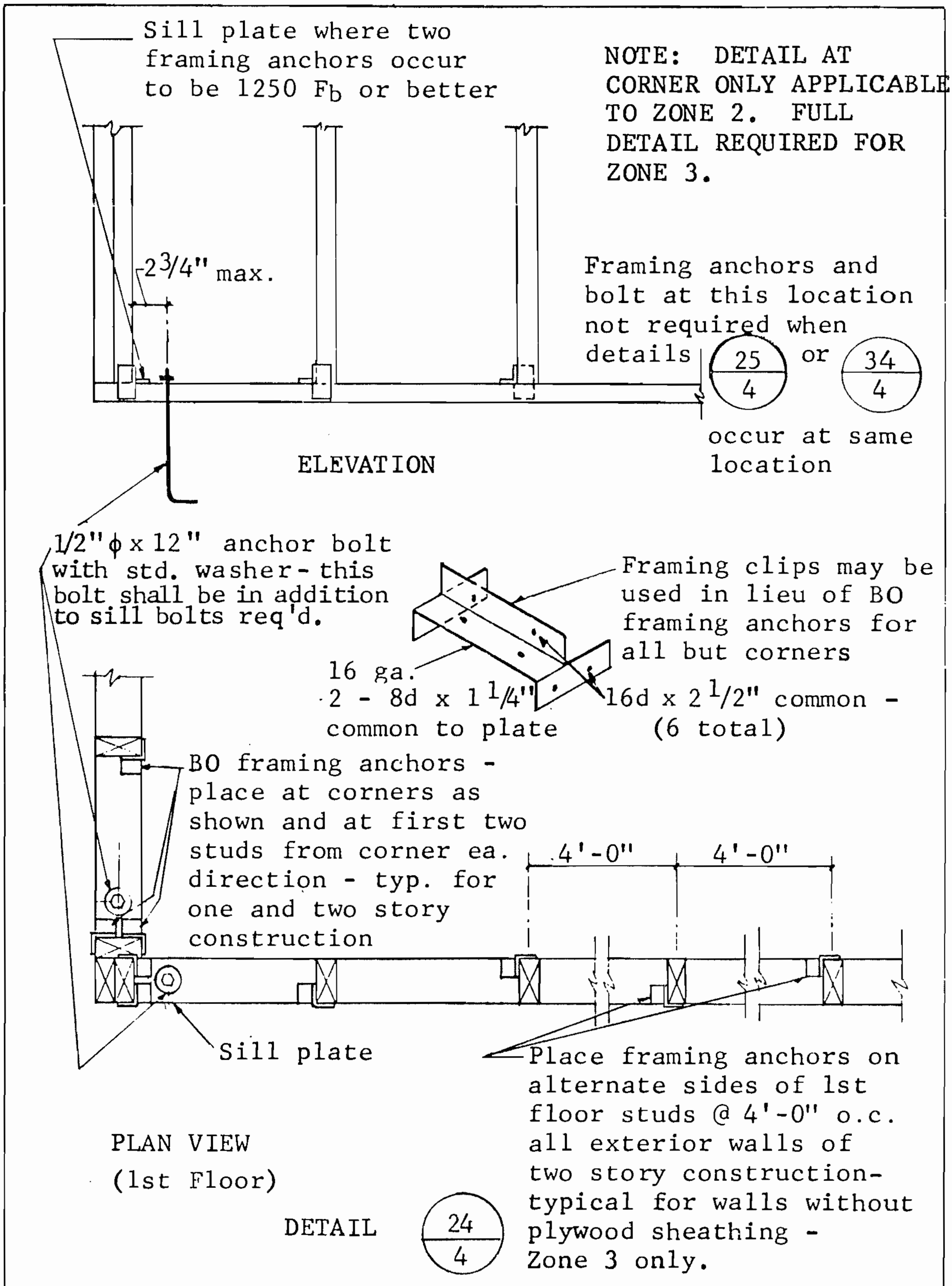
DETAIL 23/4

STANDARD FRAMING ANCHOR USE

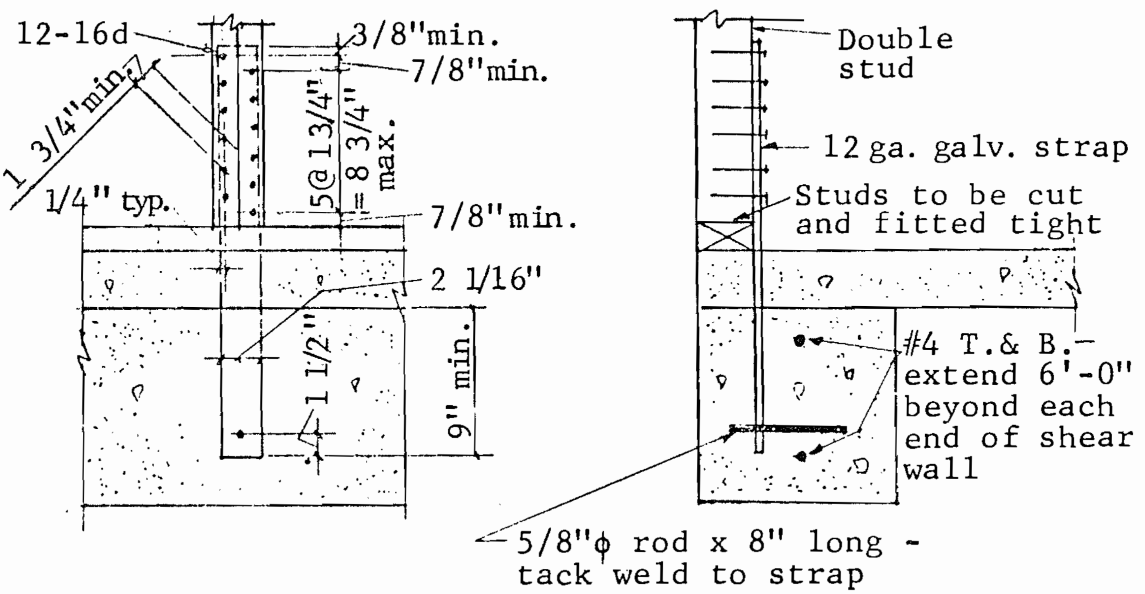
Minimum thickness = 18 gauge
Minimum nail size = 11 gauge (approx. 8d) x 1¼"

Allowable loads shown are for seismic conditions only.
See manufacturer's catalog for all other conditions of use.

(Values shown established from commercial catalogs. Where nail values govern, allowable loads have been increased one third for short term loading).



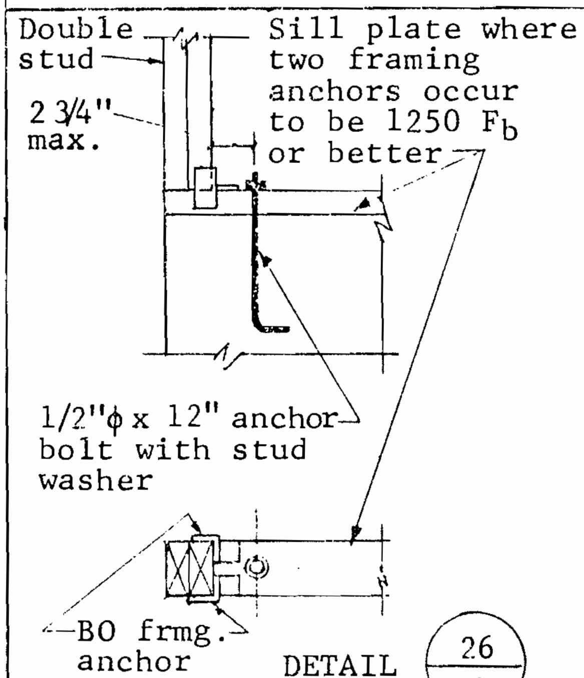
FASTENING STUDS TO SOLE PLATES



DETAIL 25/4

TYPICAL STRAP HOLD-DOWN

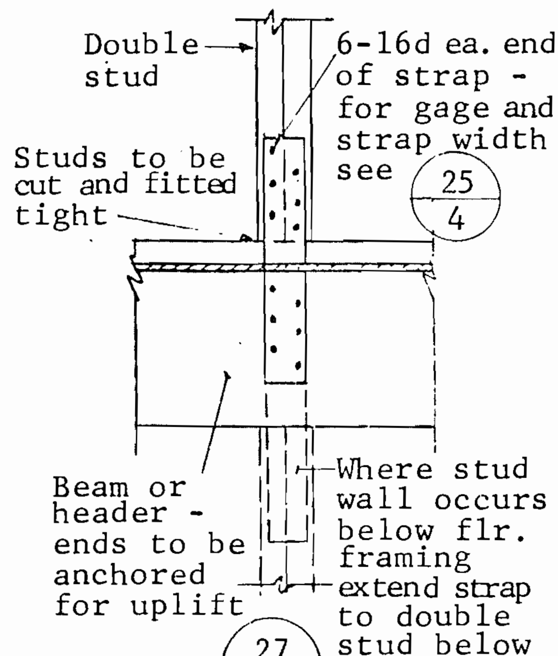
Capacity = 1700 lbs



DETAIL 26/4

FRAMING ANCHOR HOLD-DOWN

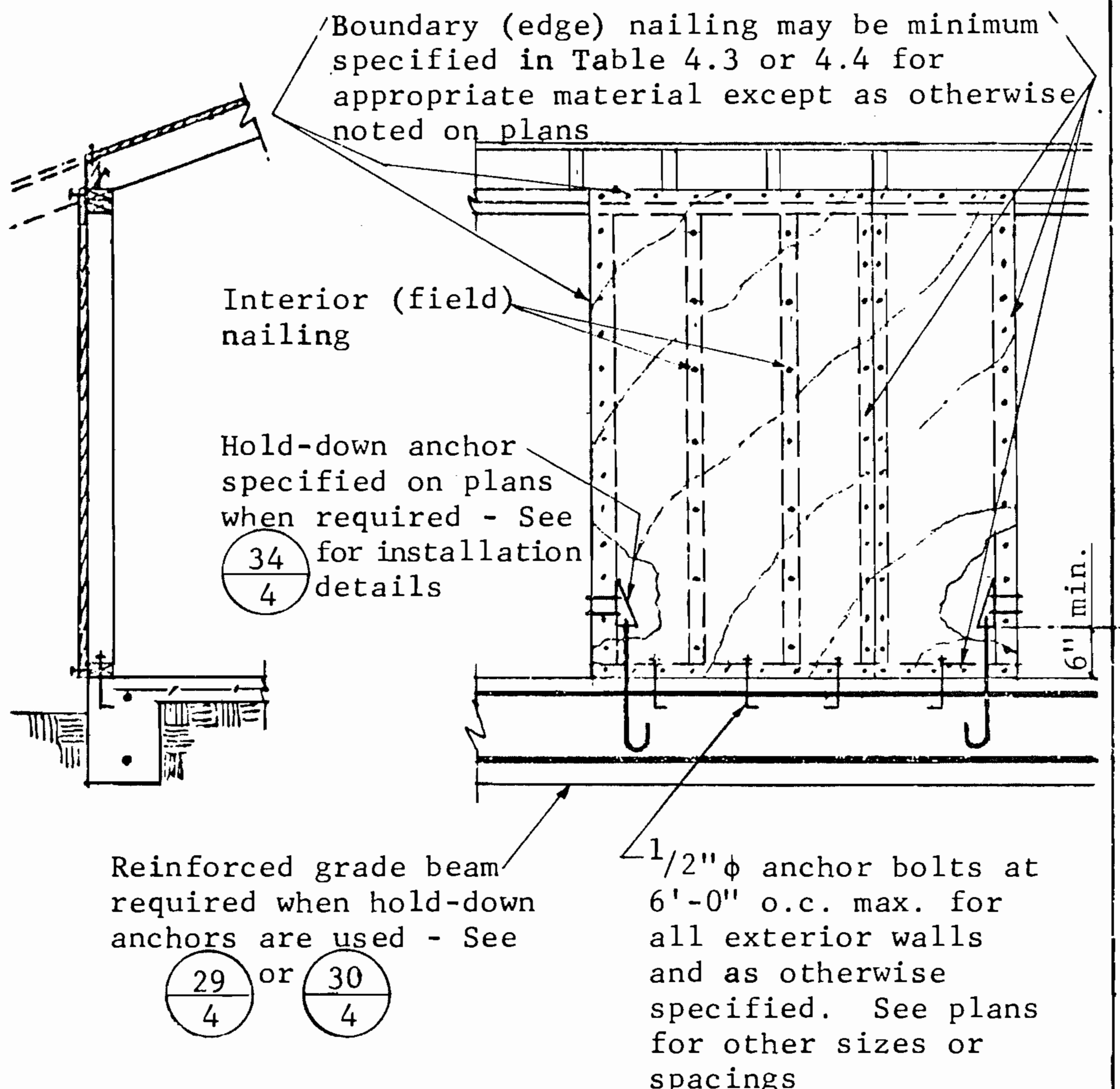
Capacity [1 Anchor = 400 lbs; 2 Anchors = 800 lbs]



DETAIL 27/4

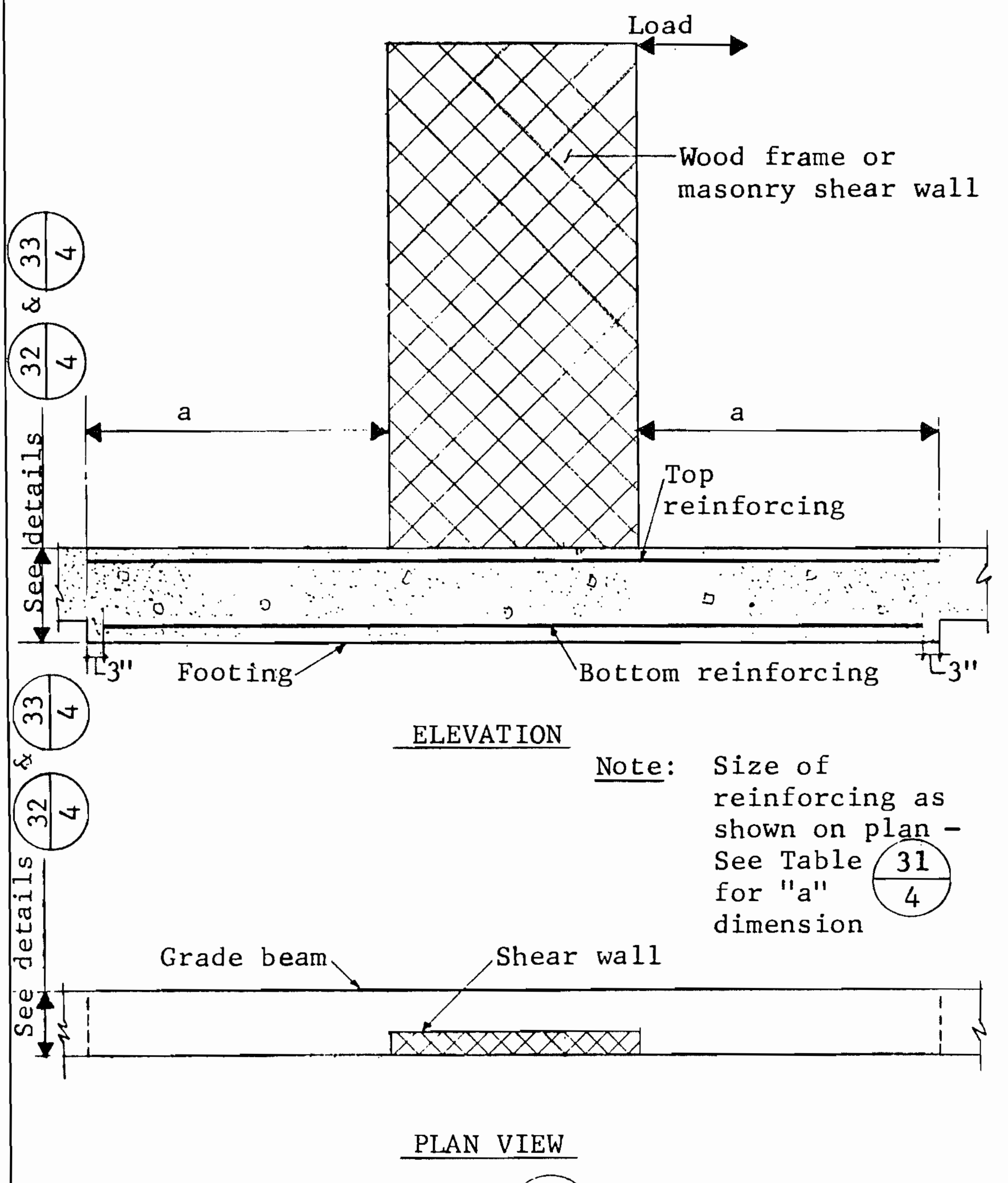
STRAP HOLD-DOWN

Capacity = 800 lbs



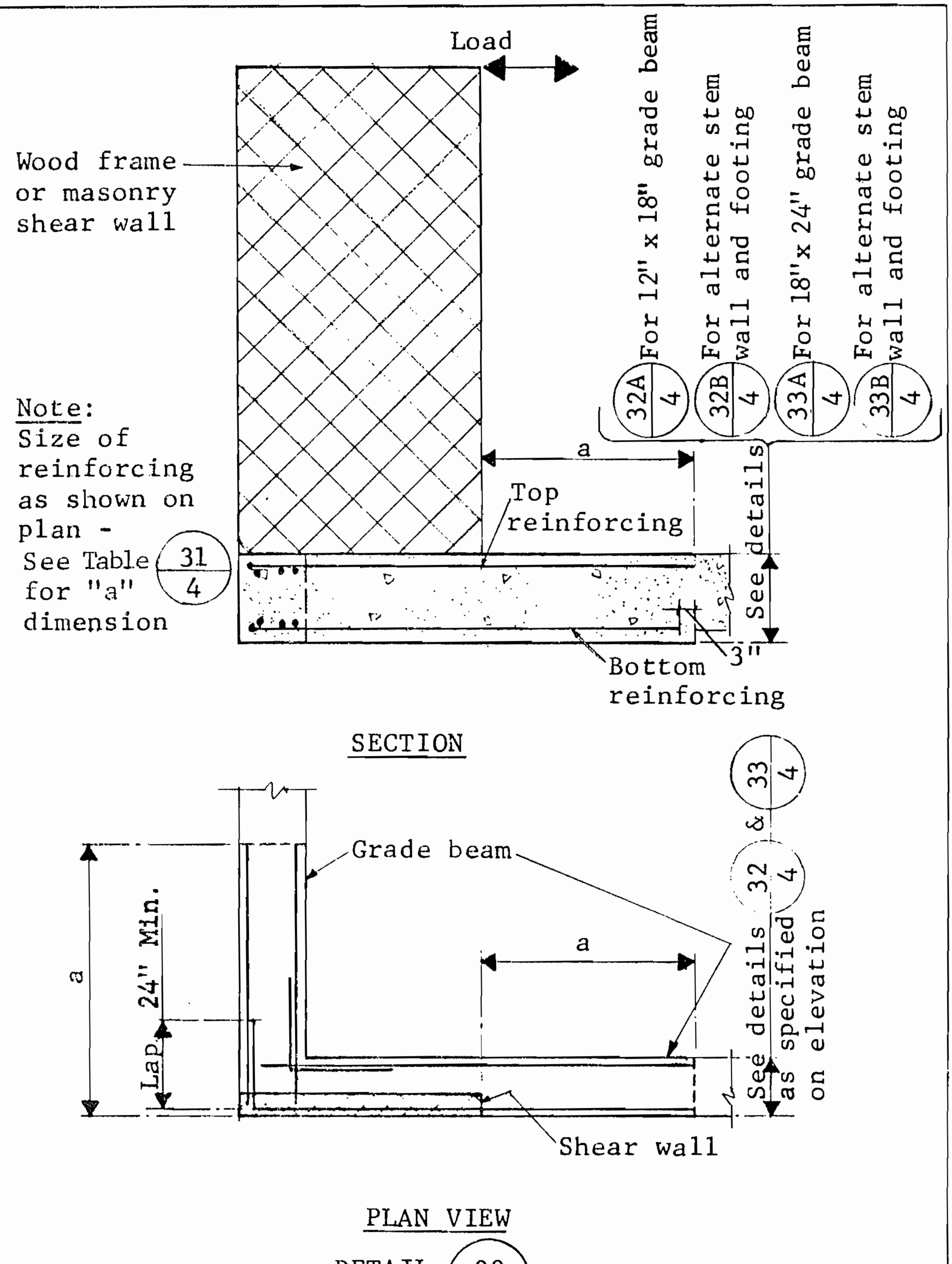
DETAIL 28/4

TYPICAL SHEAR WALL INSTALLATION



DETAIL 29/4

TYPICAL GRADE BEAM AT SHEAR WALL

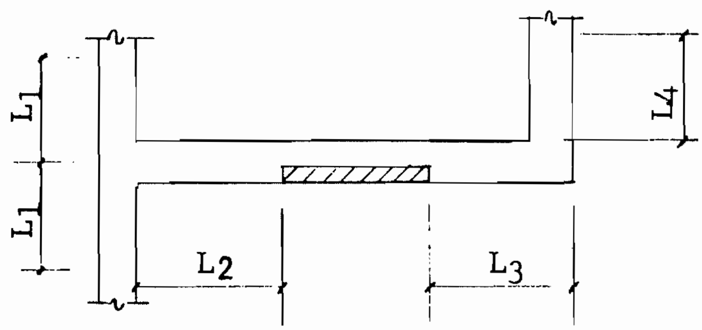


DETAIL 30/4

TYPICAL GRADE BEAM AND SHEAR WALL AT CORNER

TABLE 4.2 LENGTH OF REINFORCING BEYOND END OF SHEAR WALL

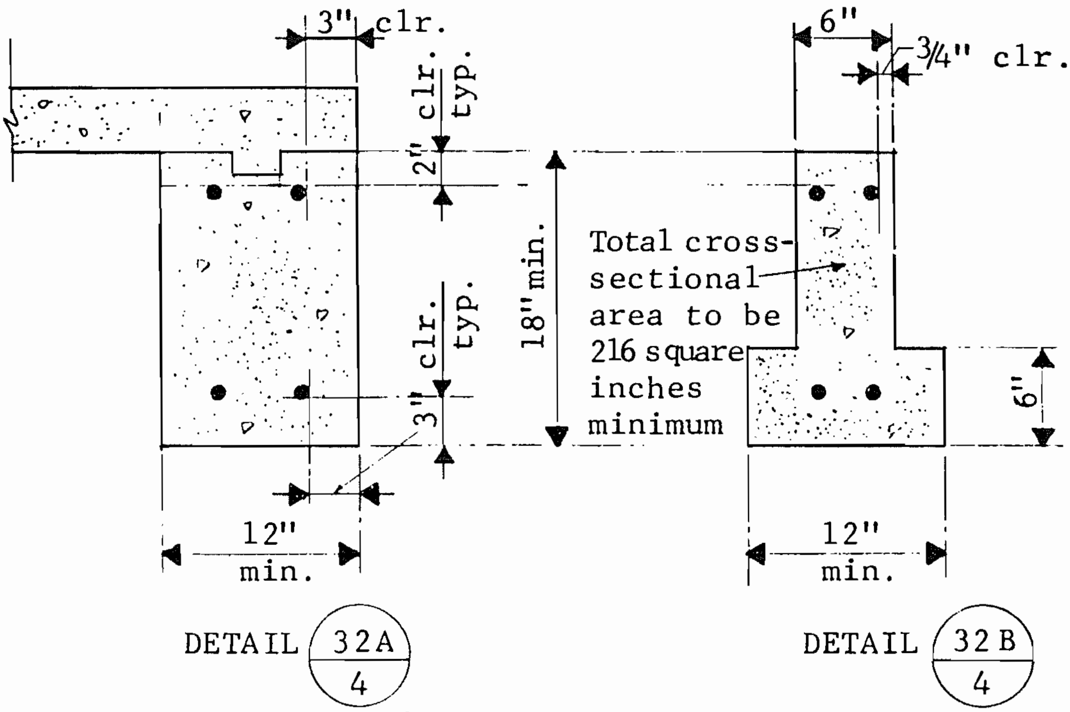
FOOTING SIZE	TOP AND BOTTOM REINFORCING	"a"
Width x Depth	Grade 40 Reinf.	Length
12" x 18" Detail 32A/4 or 32B/4	1-#4	7'-8"
	1-#5	9'-6"
	2-#4	11'-0"
	2-#5	13'-4"
	2-#6	15'-9"
18" x 24" Detail 33A/4 or 33B/4	2-#5	11'-3"
	2-#6	13'-4"
	2-#8	17'-8"



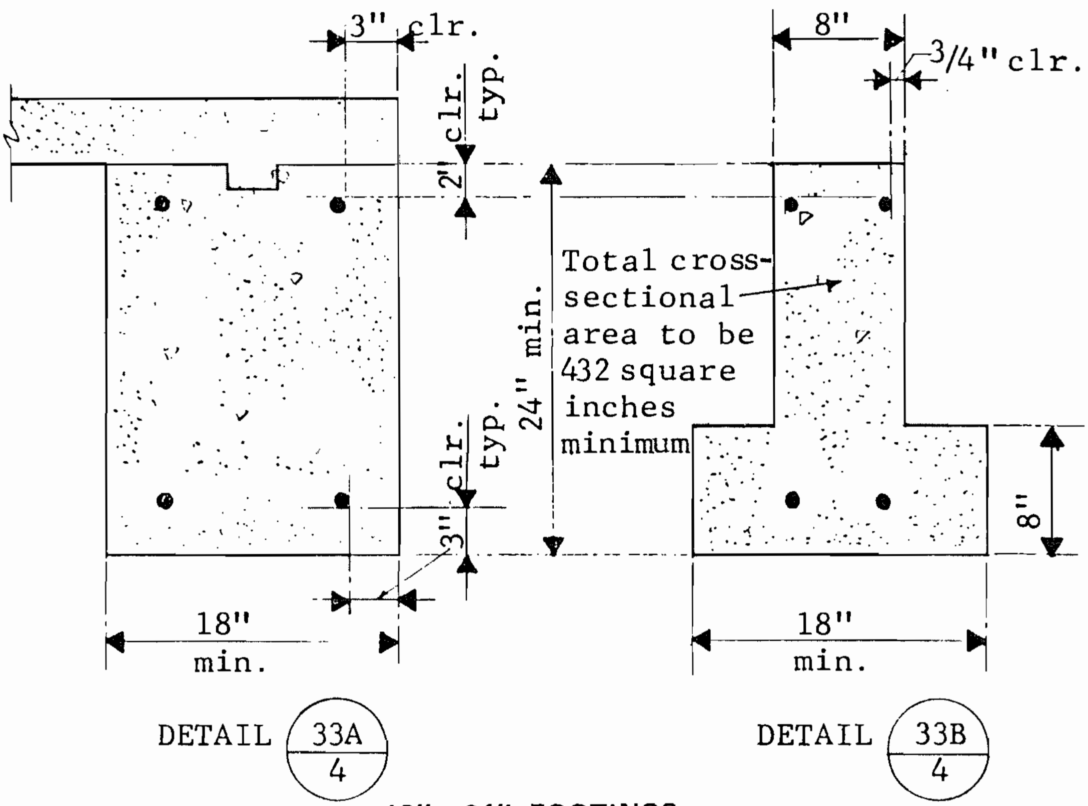
$$2L_1 + L_2 = \text{"a"}$$
$$L_3 + L_4 = \text{"a"}$$

DETAIL 31/4

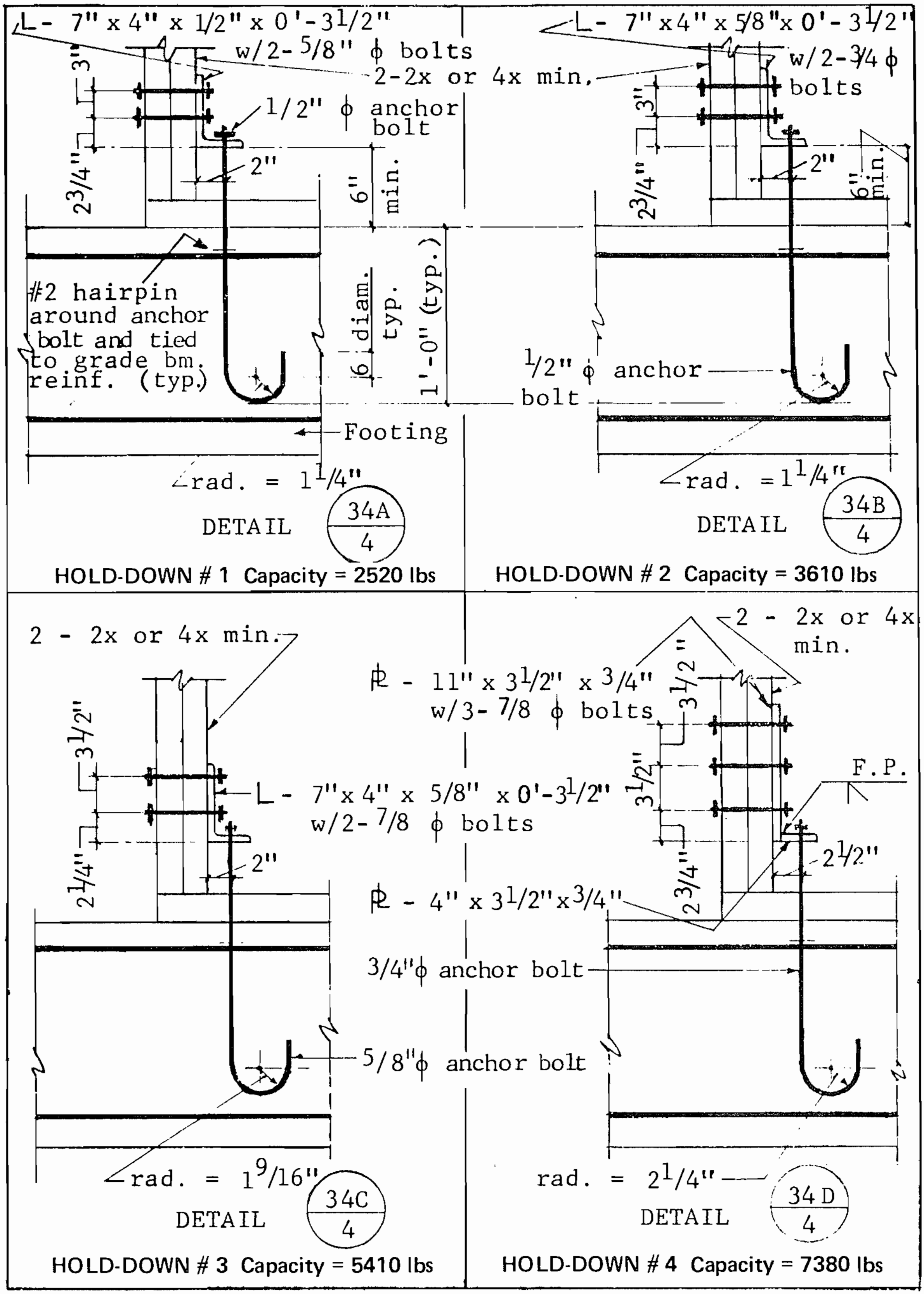
SPECIAL CASES FOR DIMENSION "a"



12" x 18" FOOTINGS



18" x 24" FOOTINGS

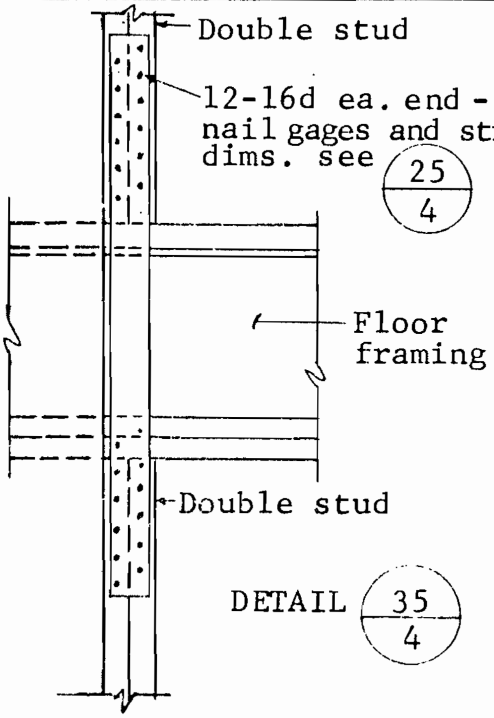


HOLD-DOWN # 1 Capacity = 2520 lbs

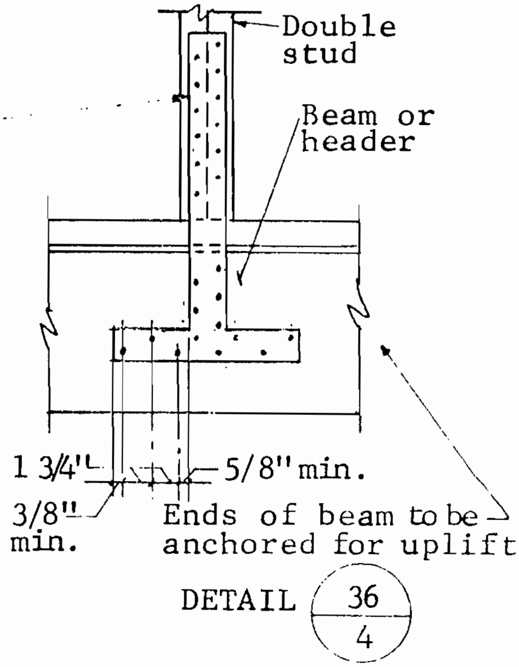
HOLD-DOWN # 2 Capacity = 3610 lbs

HOLD-DOWN # 3 Capacity = 5410 lbs

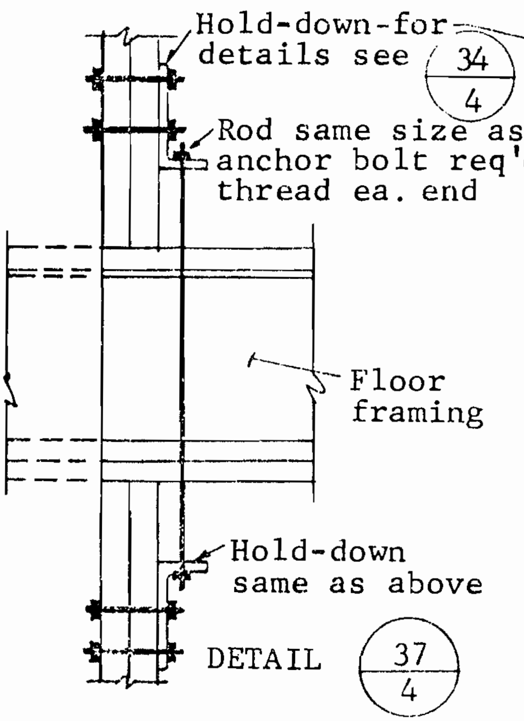
HOLD-DOWN # 4 Capacity = 7380 lbs



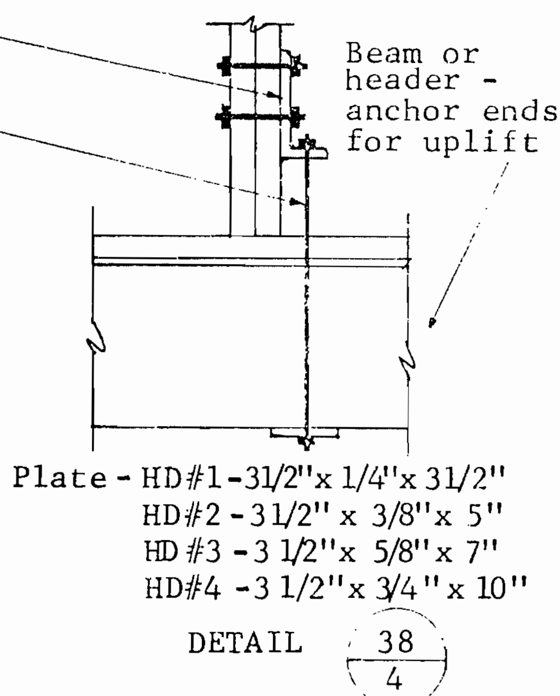
STRAP HOLD-DOWN
AT FLOOR FRAMING
Capacity = 1700 lbs



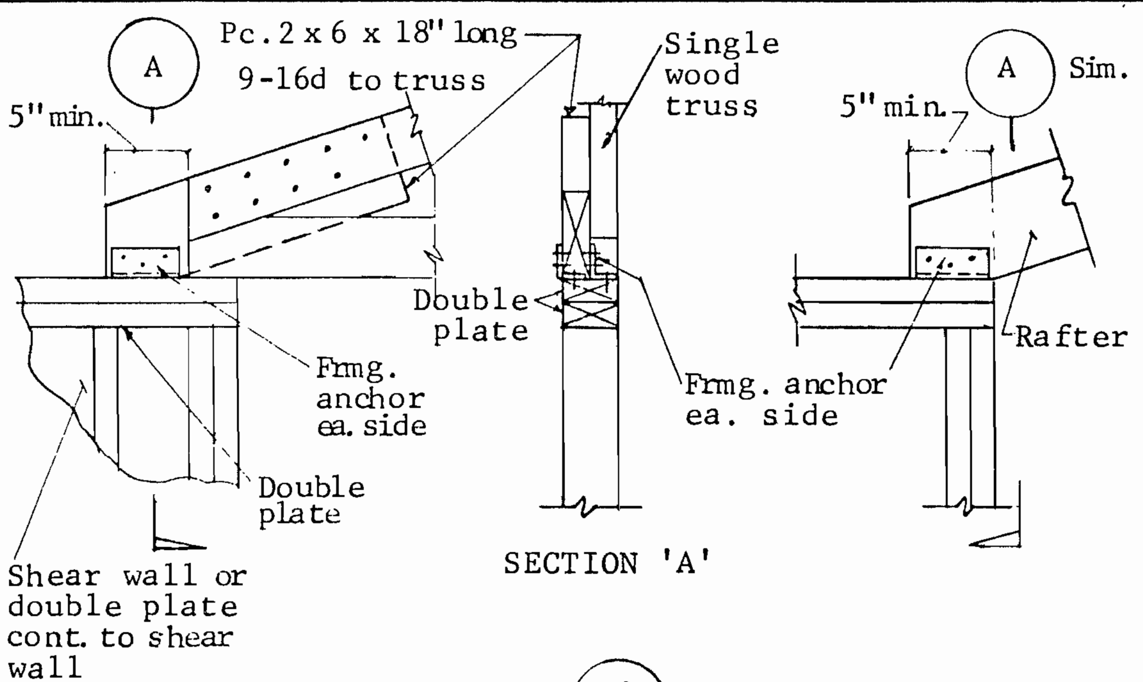
STRAP HOLD-DOWN
AT BEAM OR HEADER
Capacity = 1700 lbs



INSTALLATION OF HOLD-DOWN
AT FLOOR FRAMING



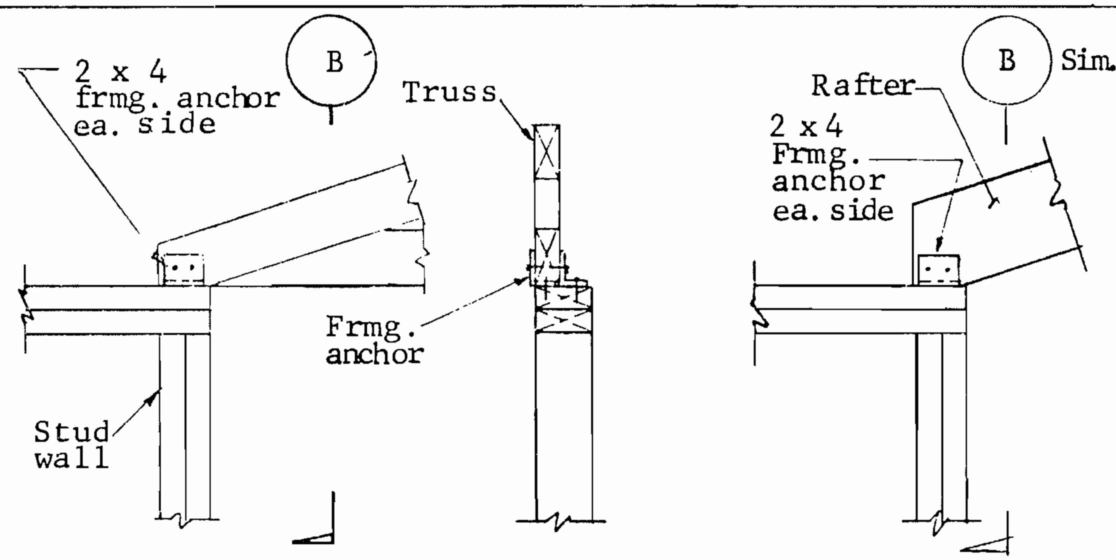
INSTALLATION OF HOLD-DOWN
TO BEAM OR HEADER



SECTION 'A'

DETAIL $\frac{39}{4}$

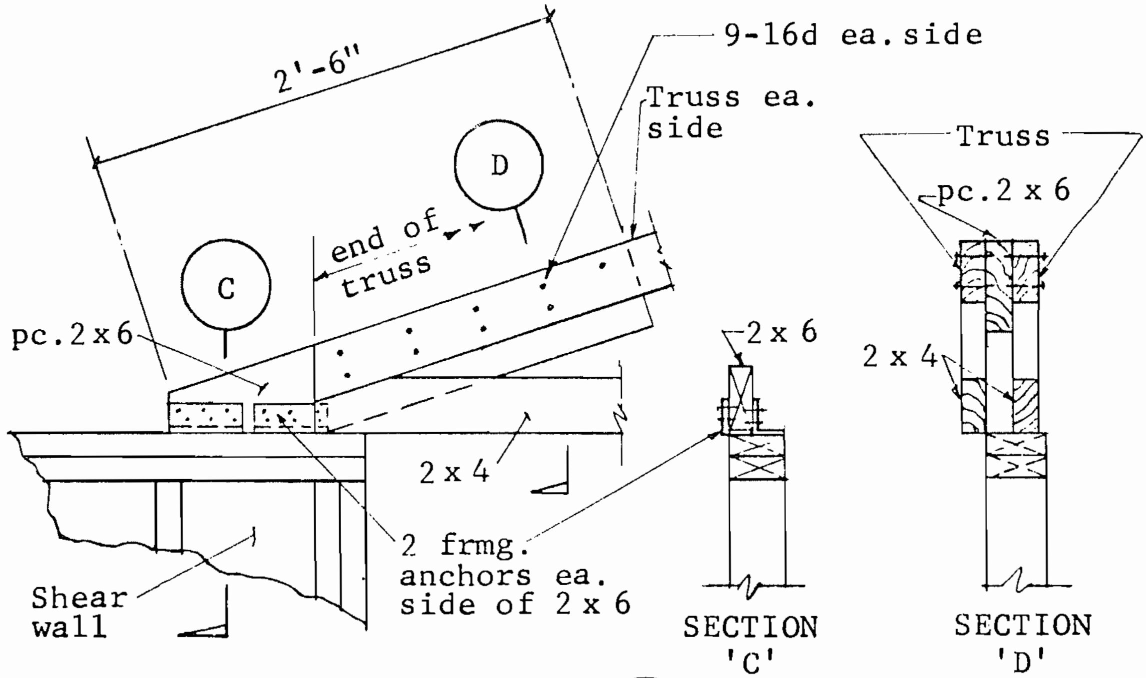
STRUT – CAPACITY = 1200 lbs



SECTION 'B'

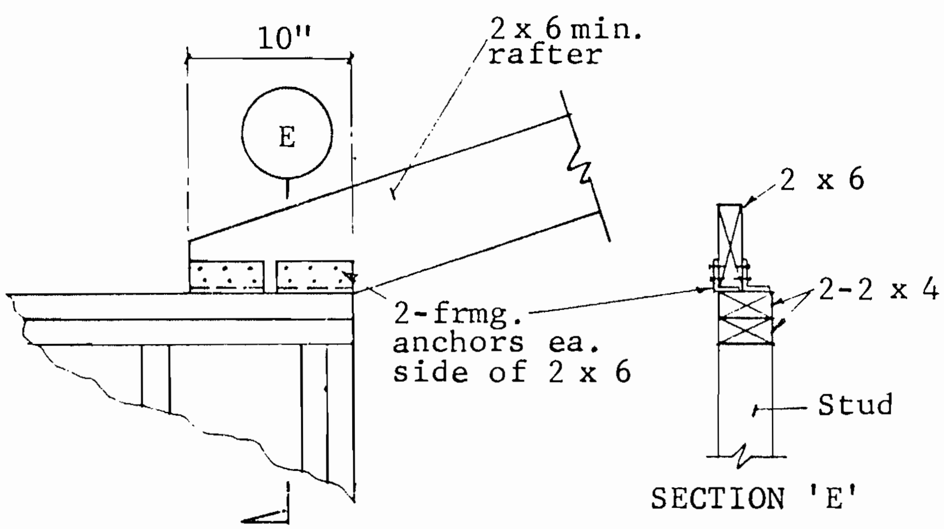
DETAIL $\frac{40}{4}$

STRUT – CAPACITY = 600 lbs



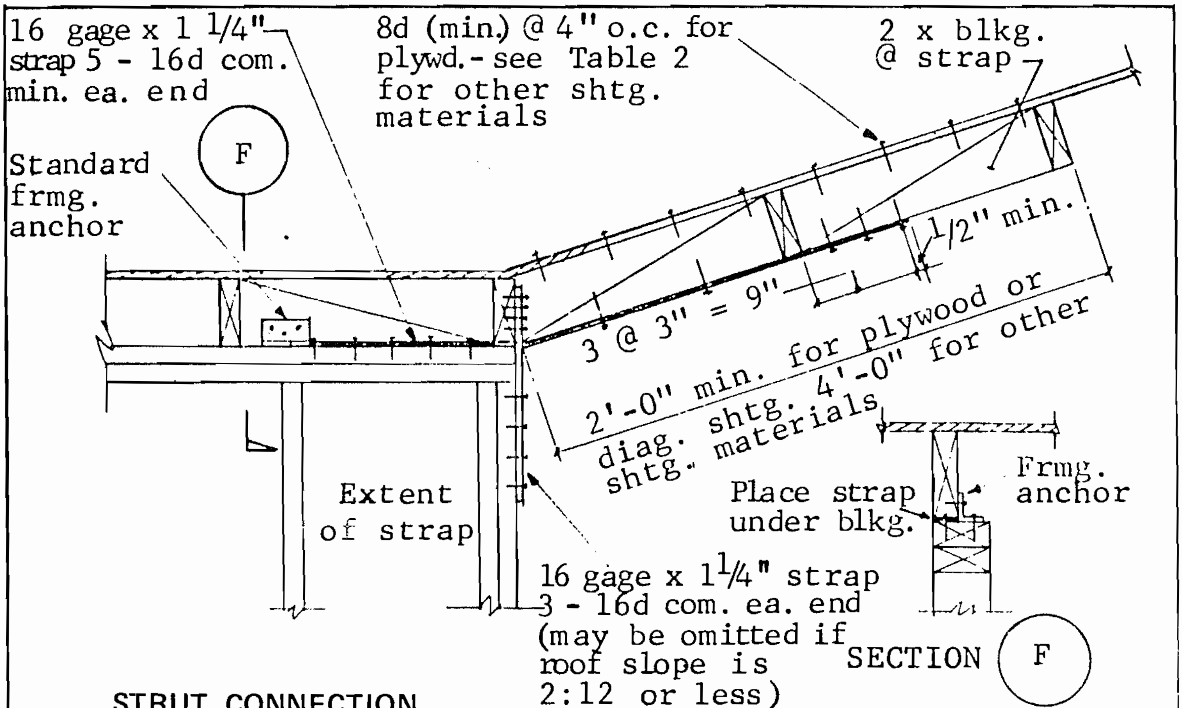
DETAIL 41A/4

STRUT – CAPACITY = 2400 lbs

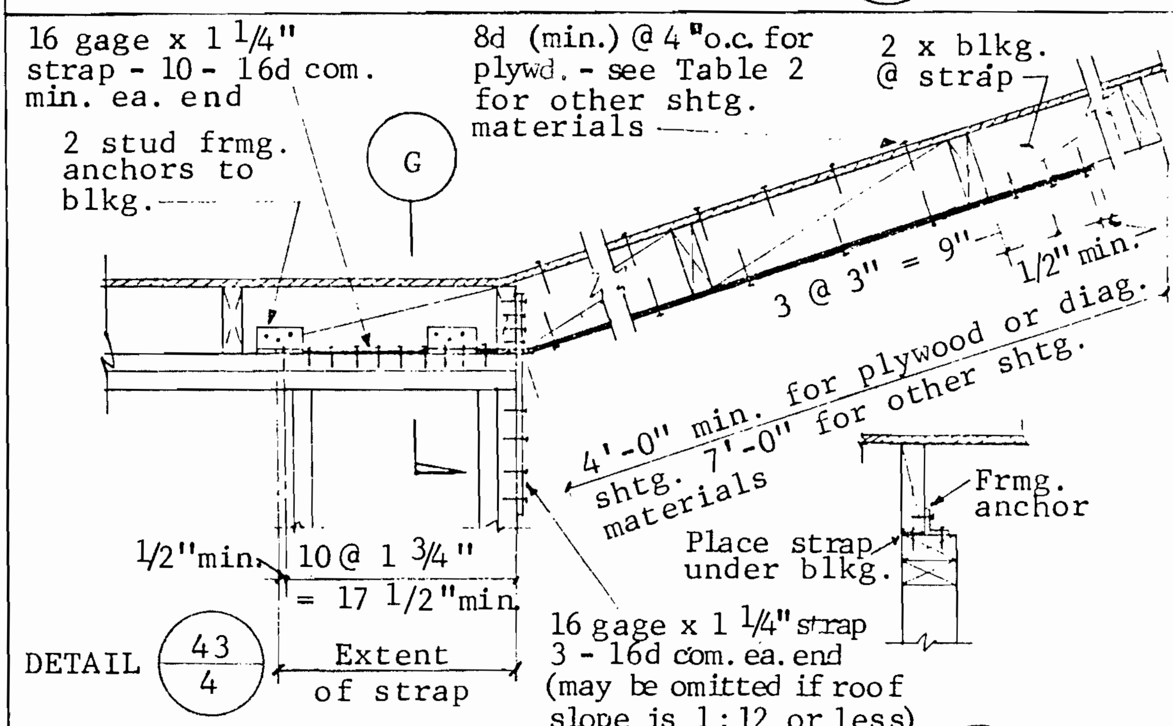


DETAIL 41B/4

STRUT – CAPACITY = 2400 lbs

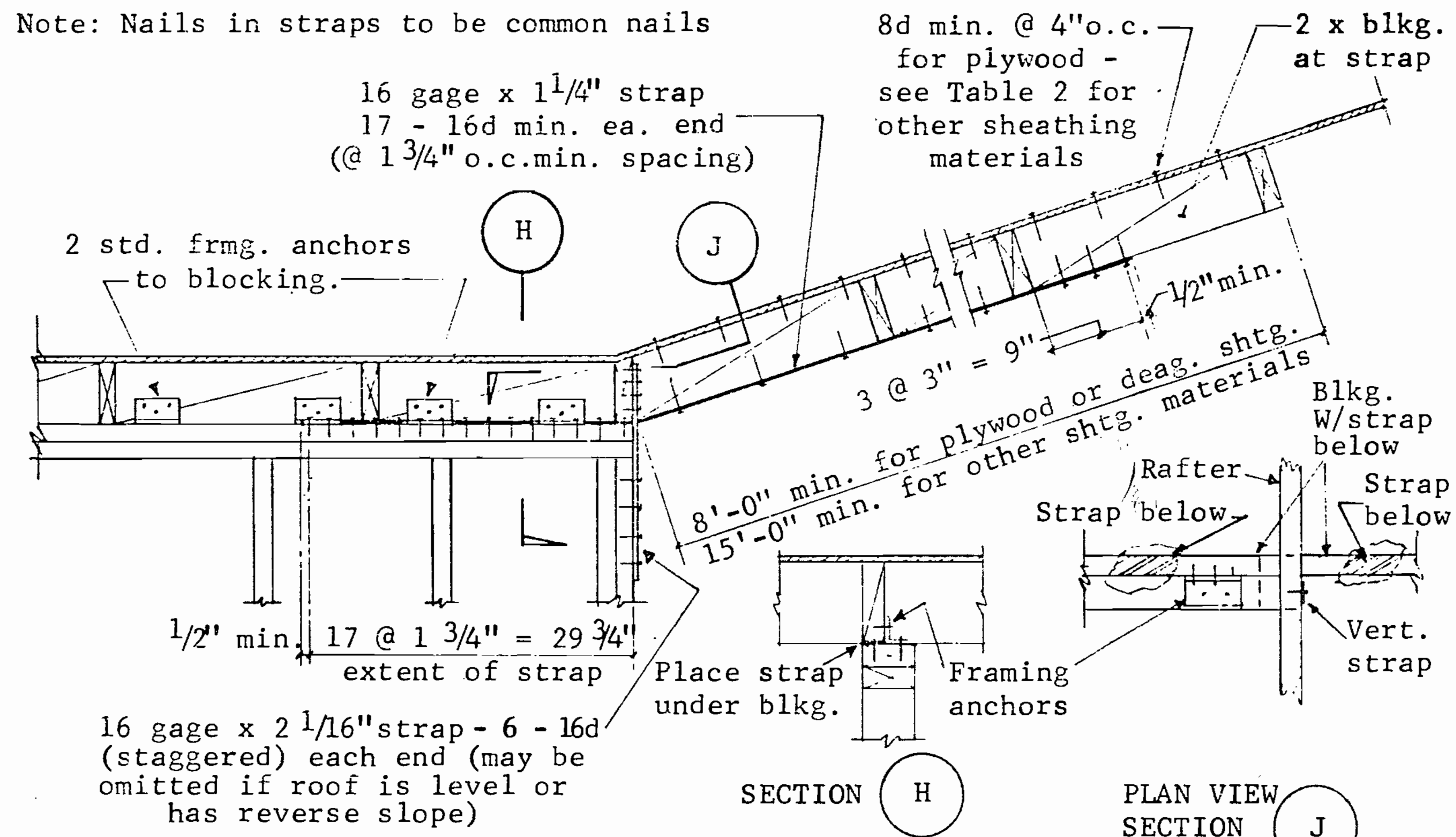


STRUT CONNECTION
RAFTERS PERPENDICULAR
Capacity = 600 lbs



STRUT CONNECTION – RAFTERS PERPENDICULAR
Capacity = 1200 lbs

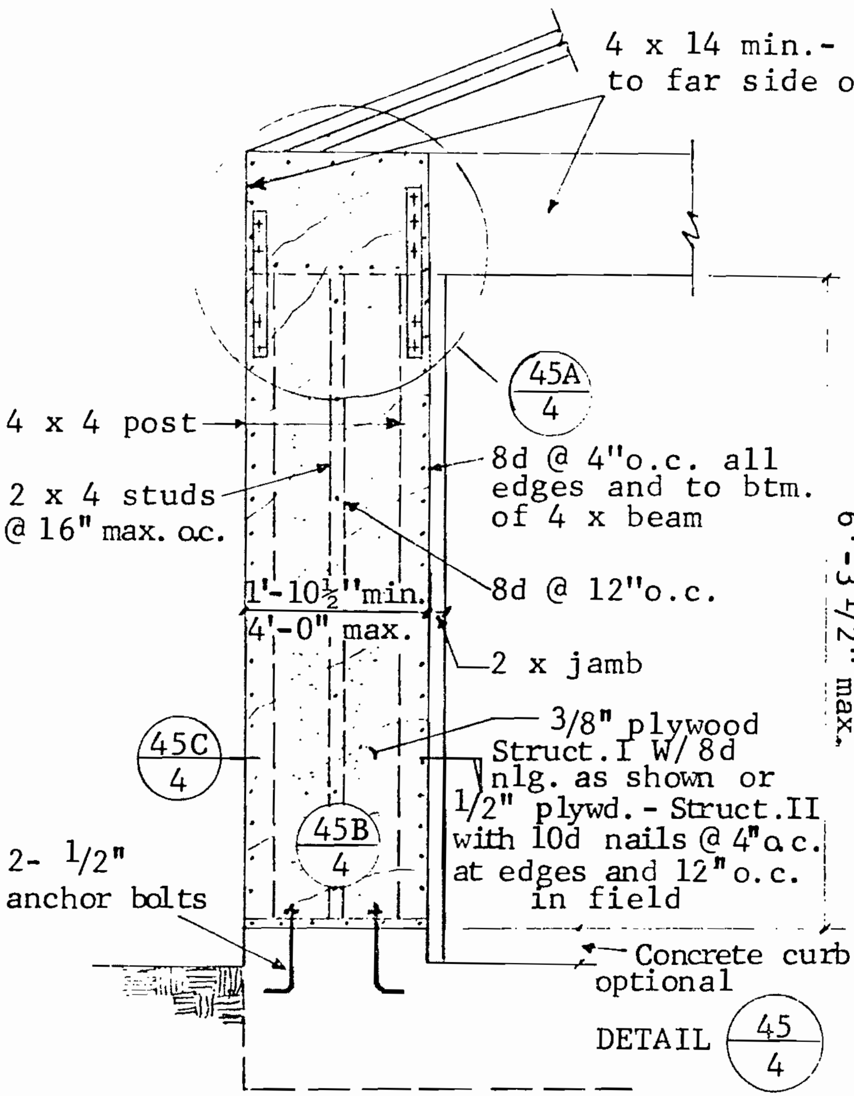
SECTION G

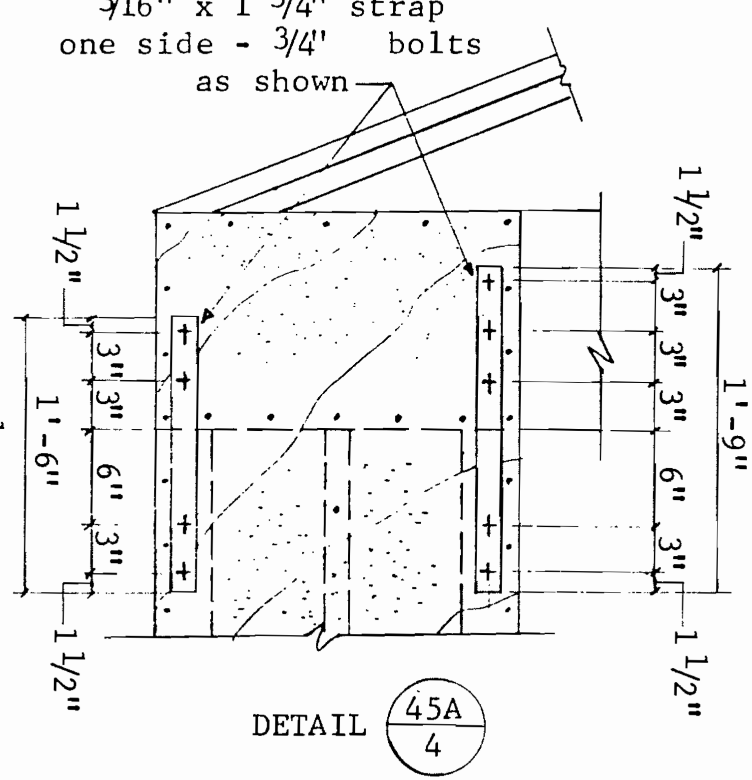


DETAIL 44/4

STRUT CONNECTION – RAFTERS PERPENDICULAR

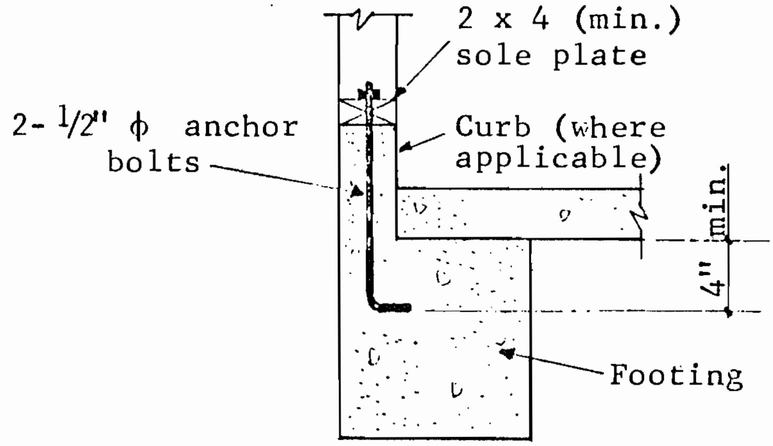
Capacity = 2400 lbs



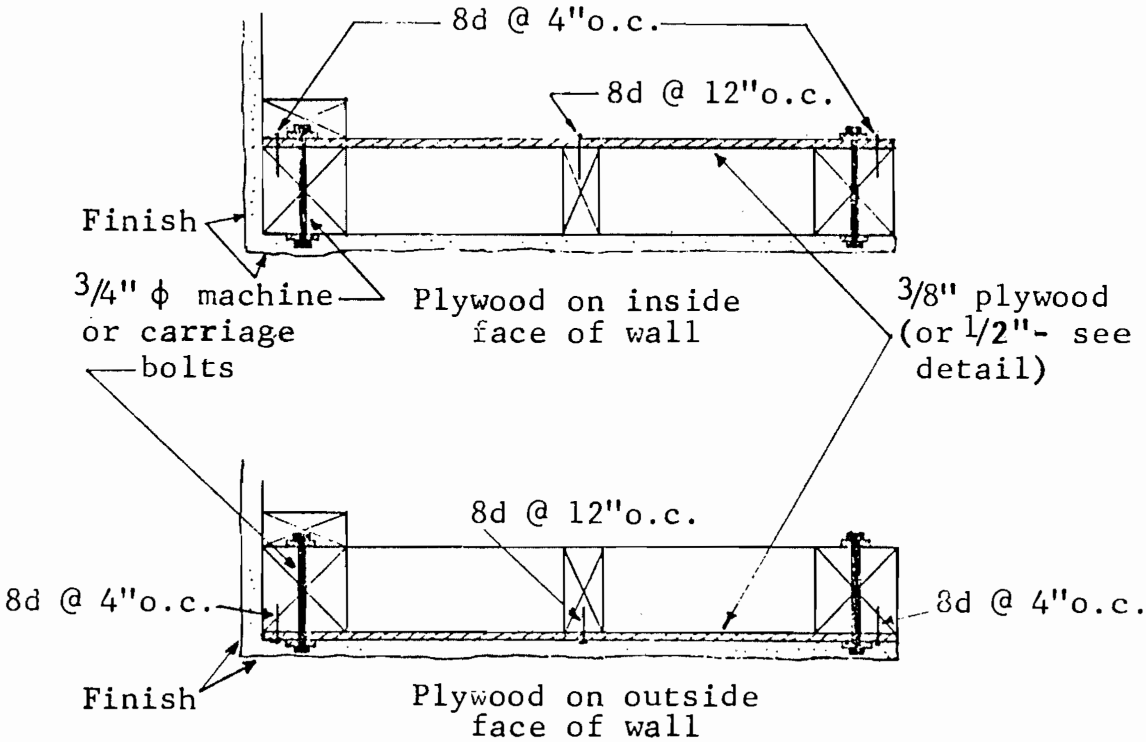


SPECIAL GARAGE FRONT WALL DETAIL
FOR SINGLE-STORY GARAGES

Zones 2 and 3 — Required each side of garage door

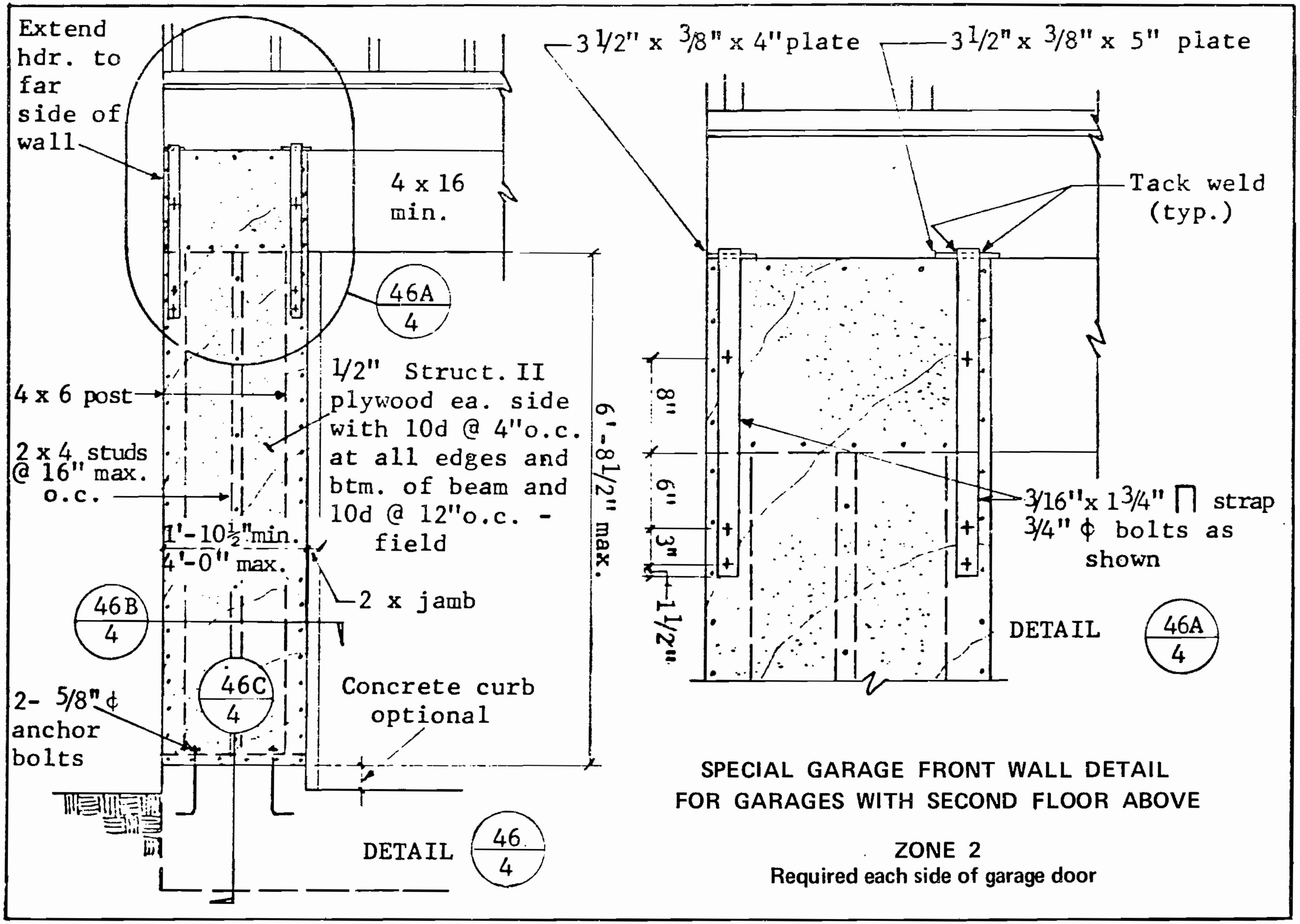


DETAIL 45B/4



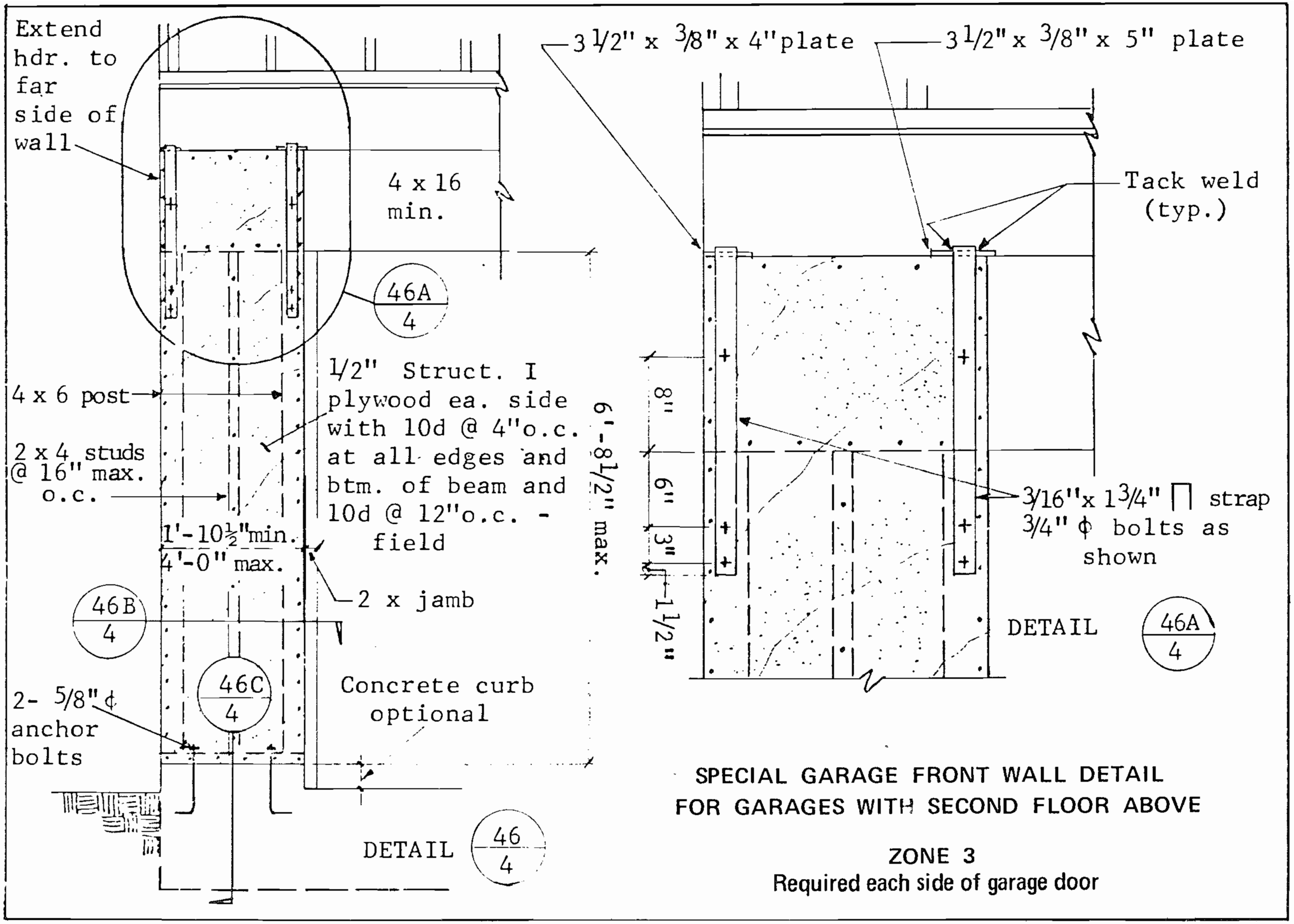
DETAIL 45C/4

Note: Plywood and straps may be on either side of wall. Plywood and straps may be on the same or opposite sides of wall.



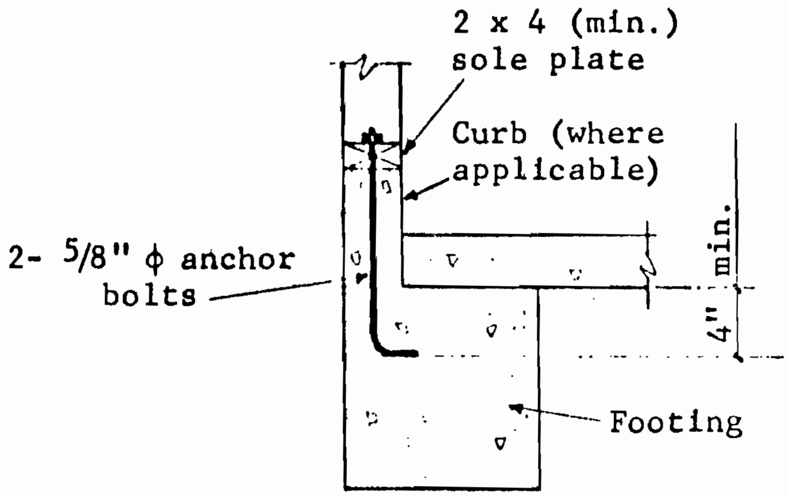
SPECIAL GARAGE FRONT WALL DETAIL
FOR GARAGES WITH SECOND FLOOR ABOVE

ZONE 2
Required each side of garage door

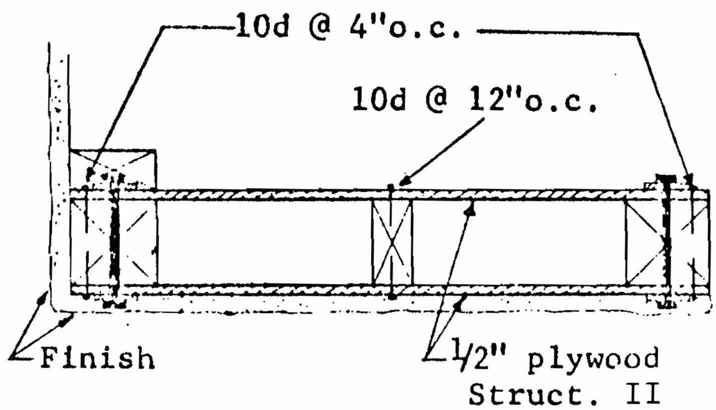


SPECIAL GARAGE FRONT WALL DETAIL
FOR GARAGES WITH SECOND FLOOR ABOVE

ZONE 3
Required each side of garage door

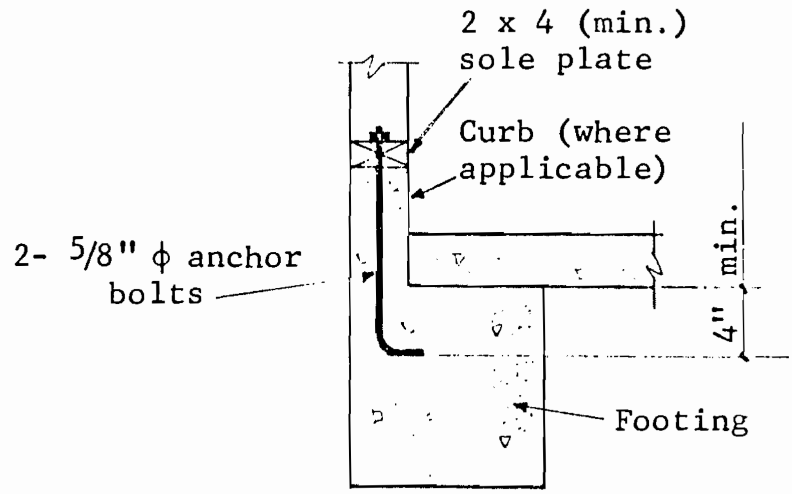


DETAIL $\frac{46B}{4}$

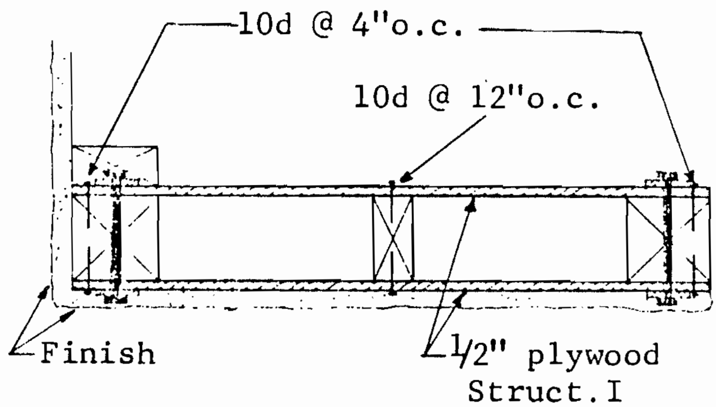


DETAIL $\frac{46C}{4}$

Note: 3/8" Structural I plywood with 8d @ 4" o.c.
may be substituted for Structural II shown.

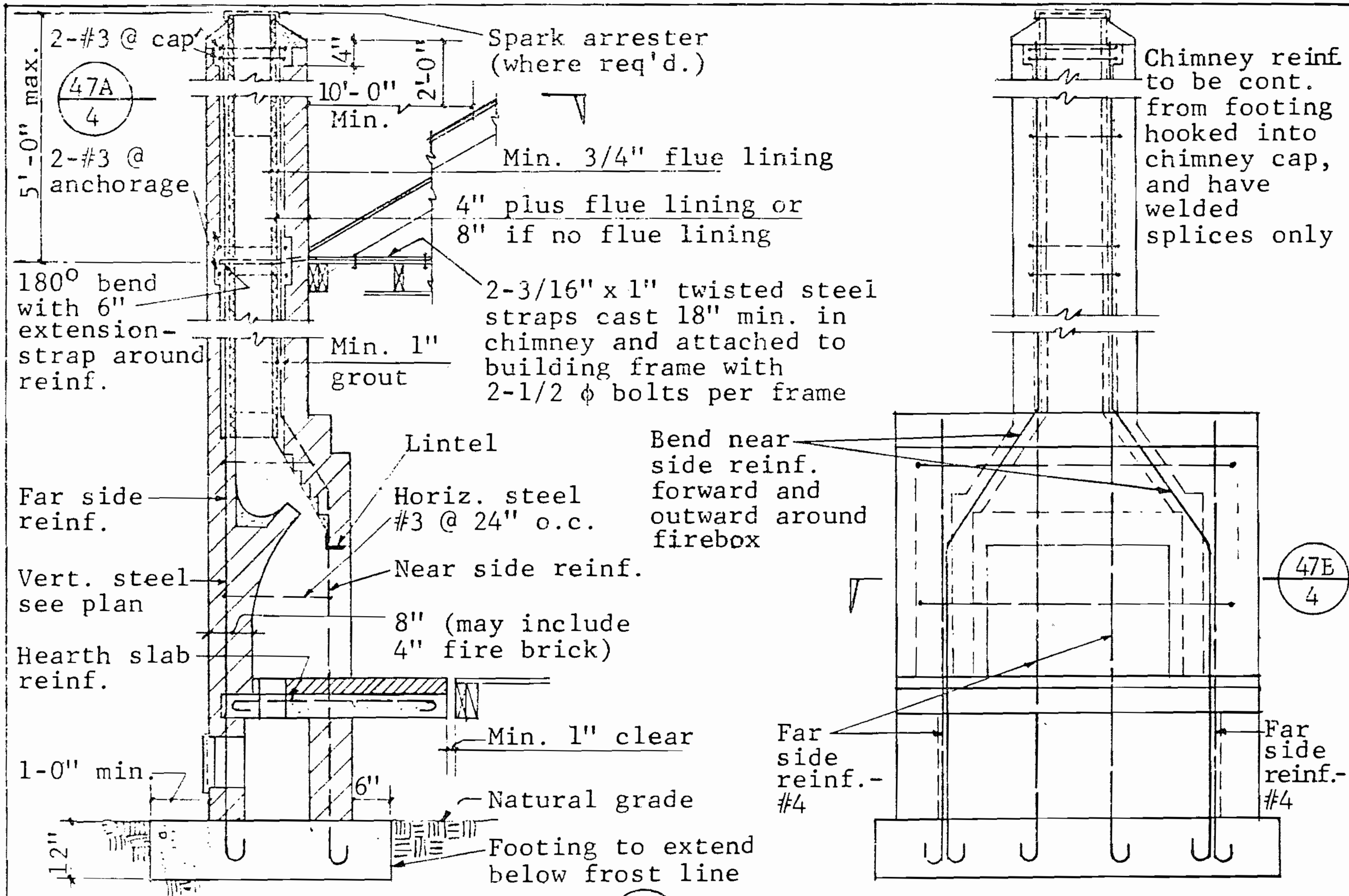


DETAIL 46B/4

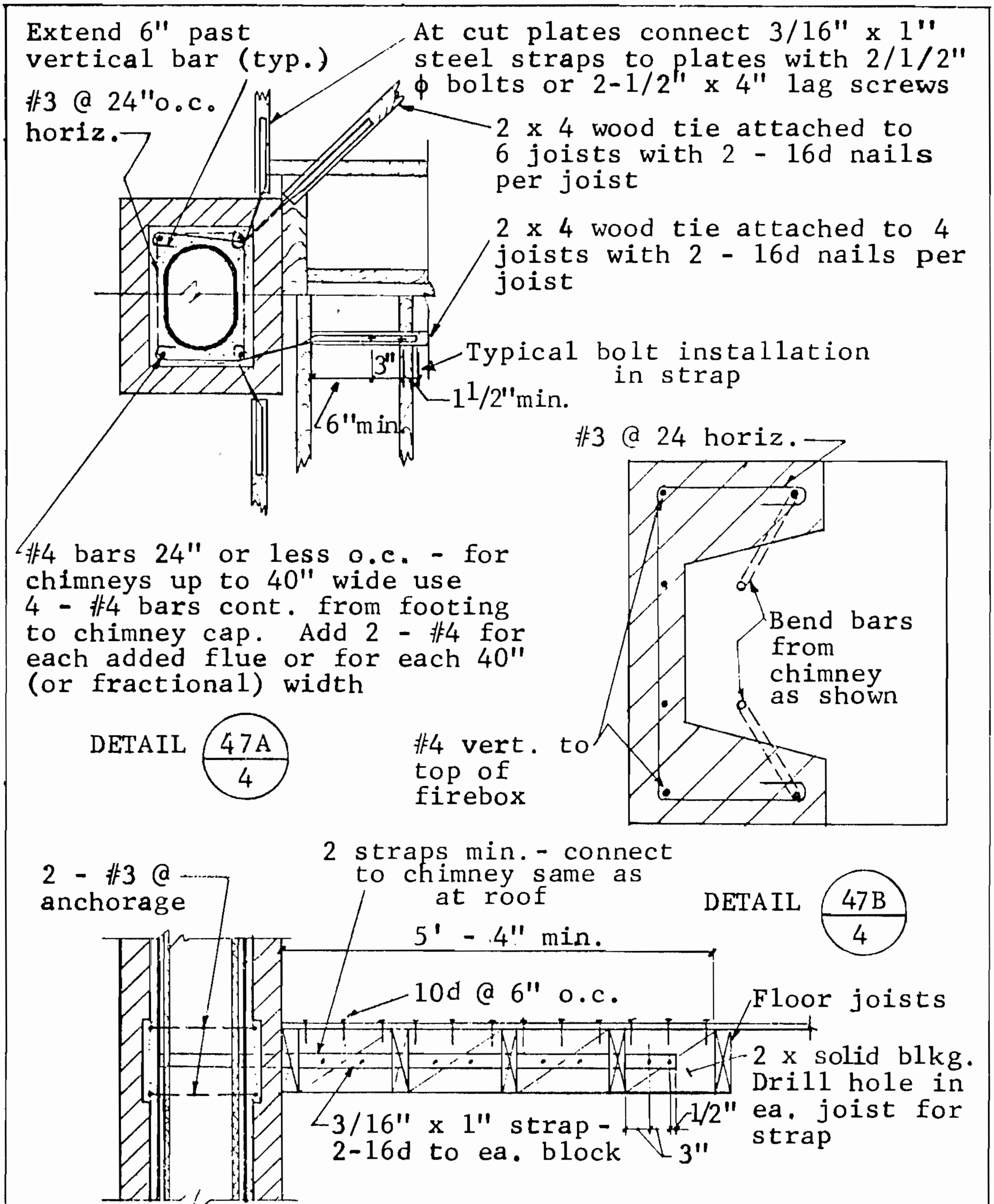


DETAIL 46C/4

Note: Structural II plywood nailed for 510#/' min. may be substituted for Structural I shown.

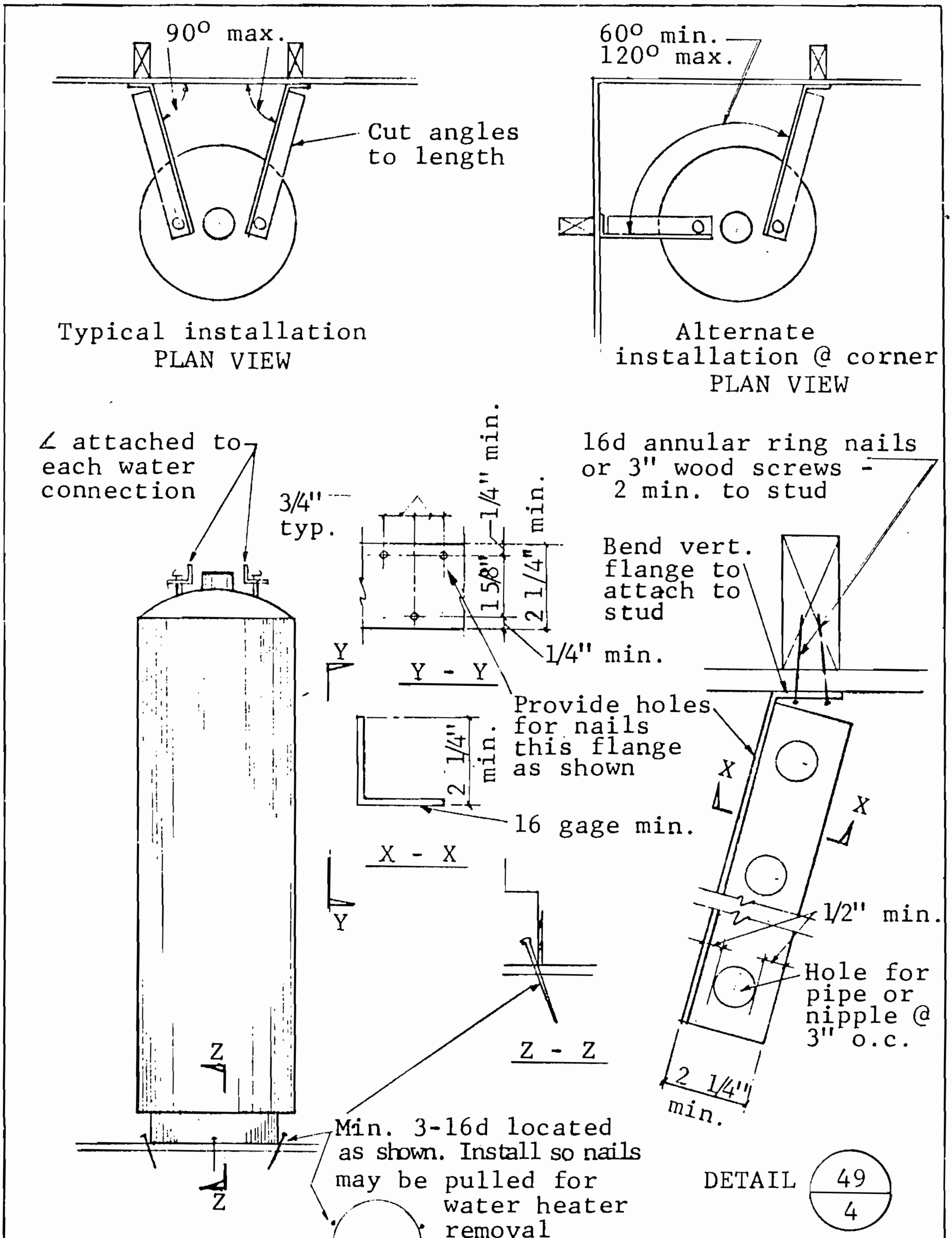


DETAIL 47/4 **TYPICAL FIREPLACE CONSTRUCTION**

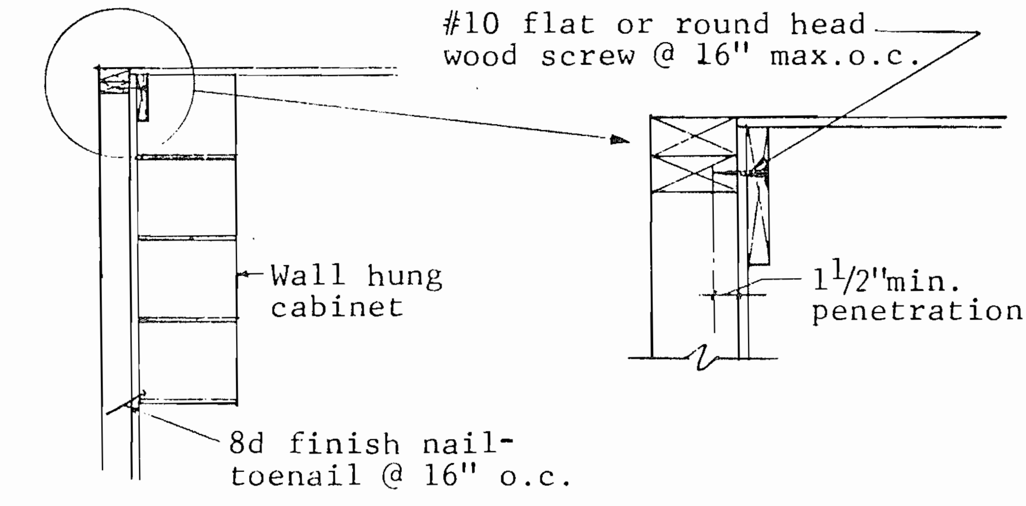


CHIMNEY TIE AT 2nd FLOOR

Notes: 1. If strap is installed at bottom of blocking or other than as shown, use std. framing anchor ea. end ea. block.
2. For joists parallel to strap connect strap to joist with 2 1/2" ϕ bolts.

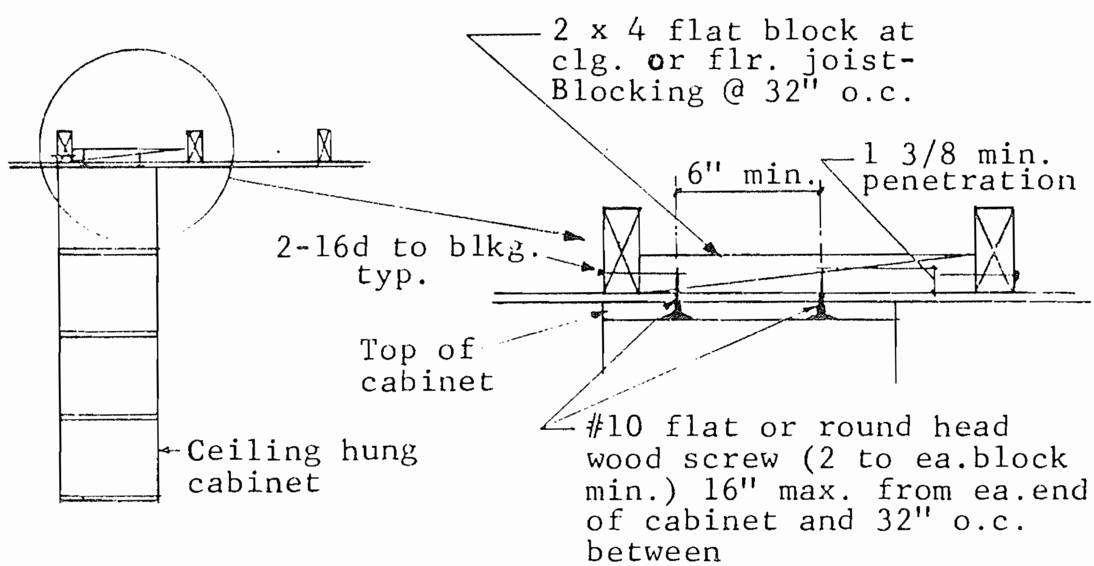


TYPICAL WATER HEATER INSTALLATION



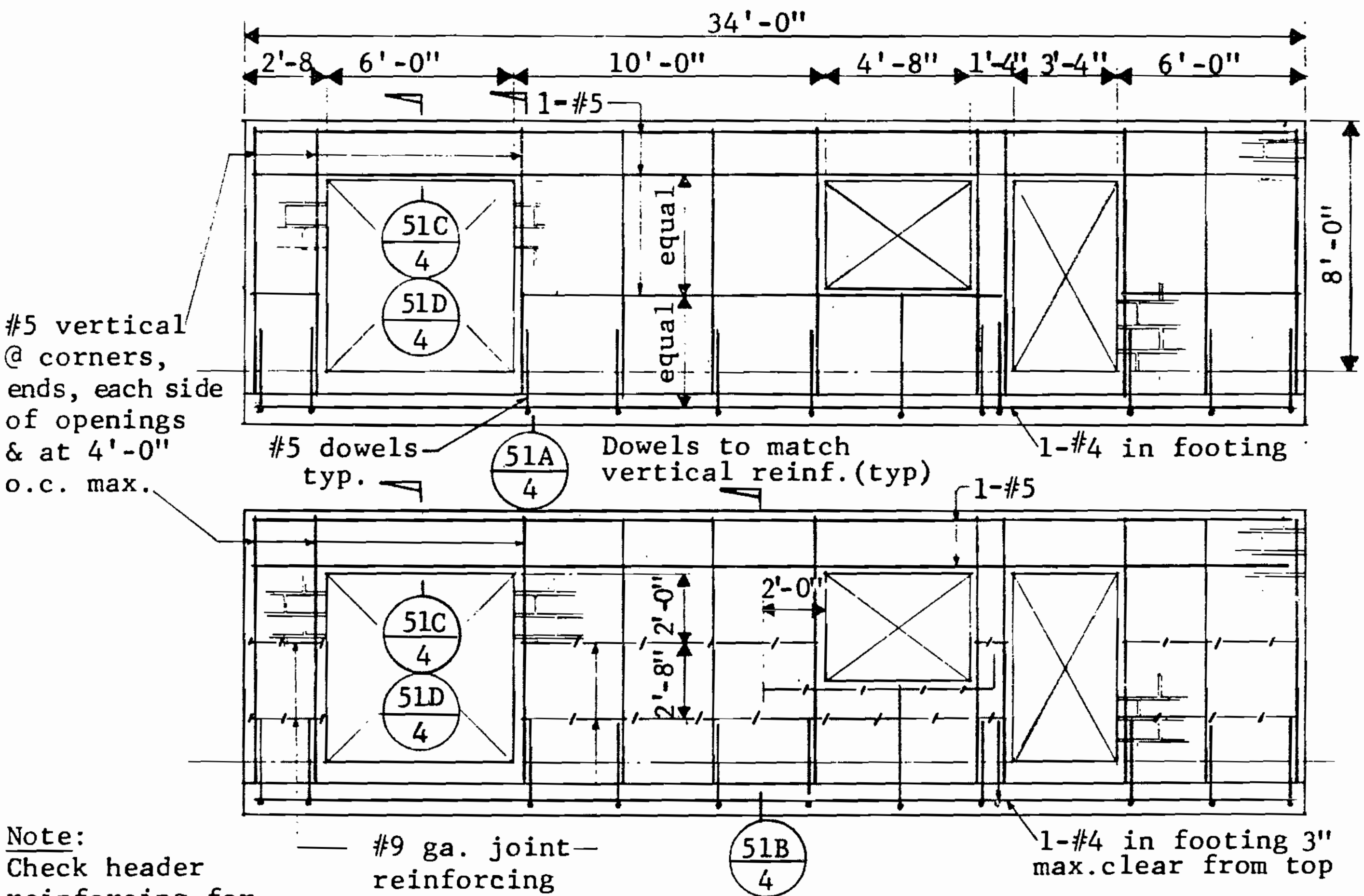
DETAIL 50A/4

MINIMUM FASTENING OF WALL-HUNG CABINETS
(Use also for full-height bookcases, cabinets, etc.)



DETAIL 50B/4

MINIMUM FASTENING OF CEILING-HUNG CABINETS
(Use also at top of free-standing bookcases, etc.)

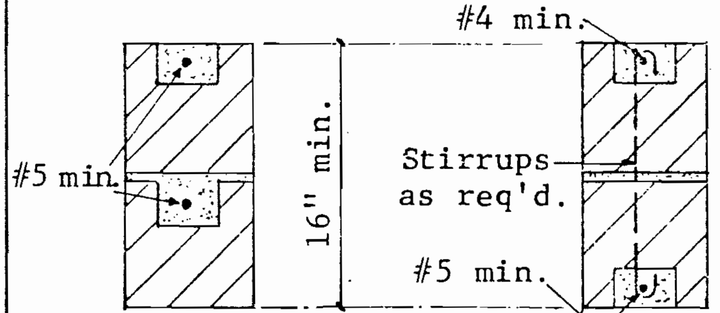


Note:
Check header reinforcing for vertical load (typ. all cases)

MINIMUM REQUIREMENTS FOR FULLY-REINFORCED 8" MASONRY WALLS

Required in Zone 3, optional in other zones

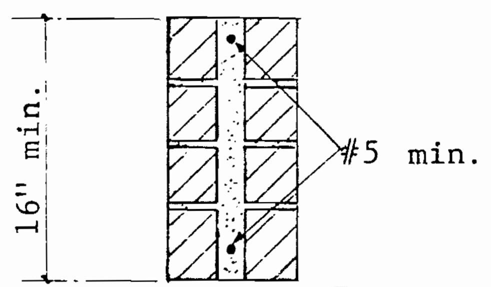
DETAIL 51/4



Short spans Longer spans

DETAIL 51C/4

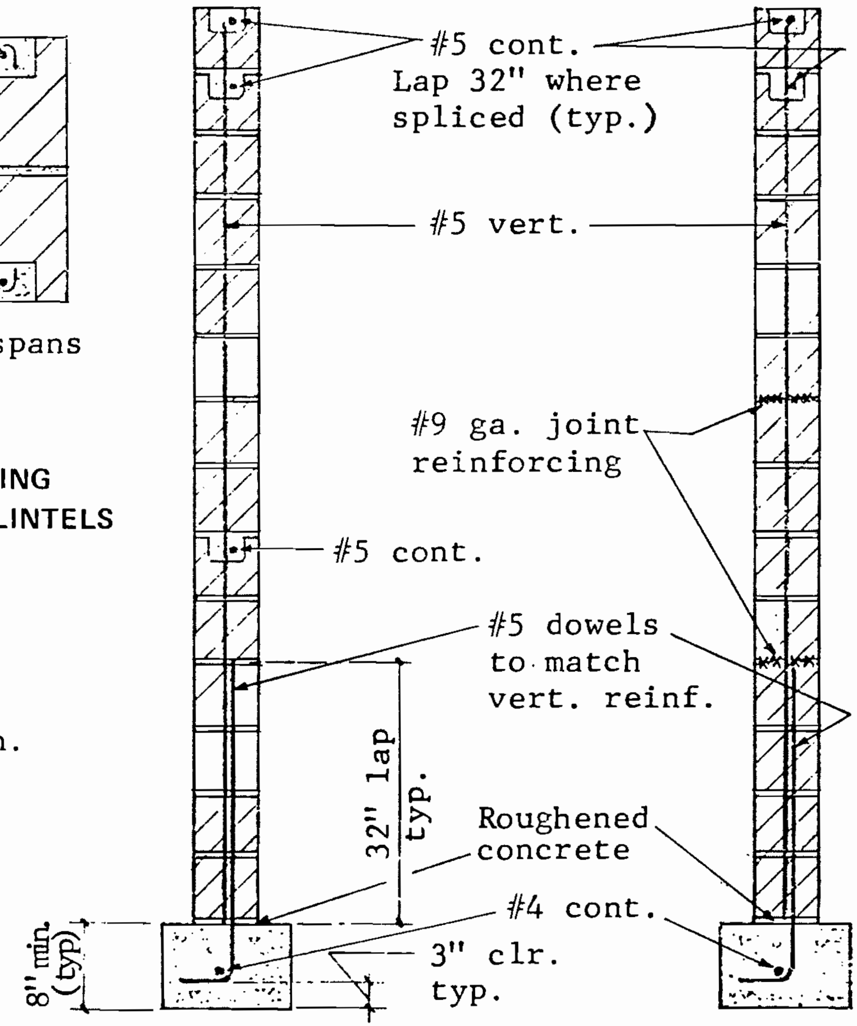
TYPICAL REINFORCING OF CONCRETE BLOCK LINTELS



DETAIL 51D/4

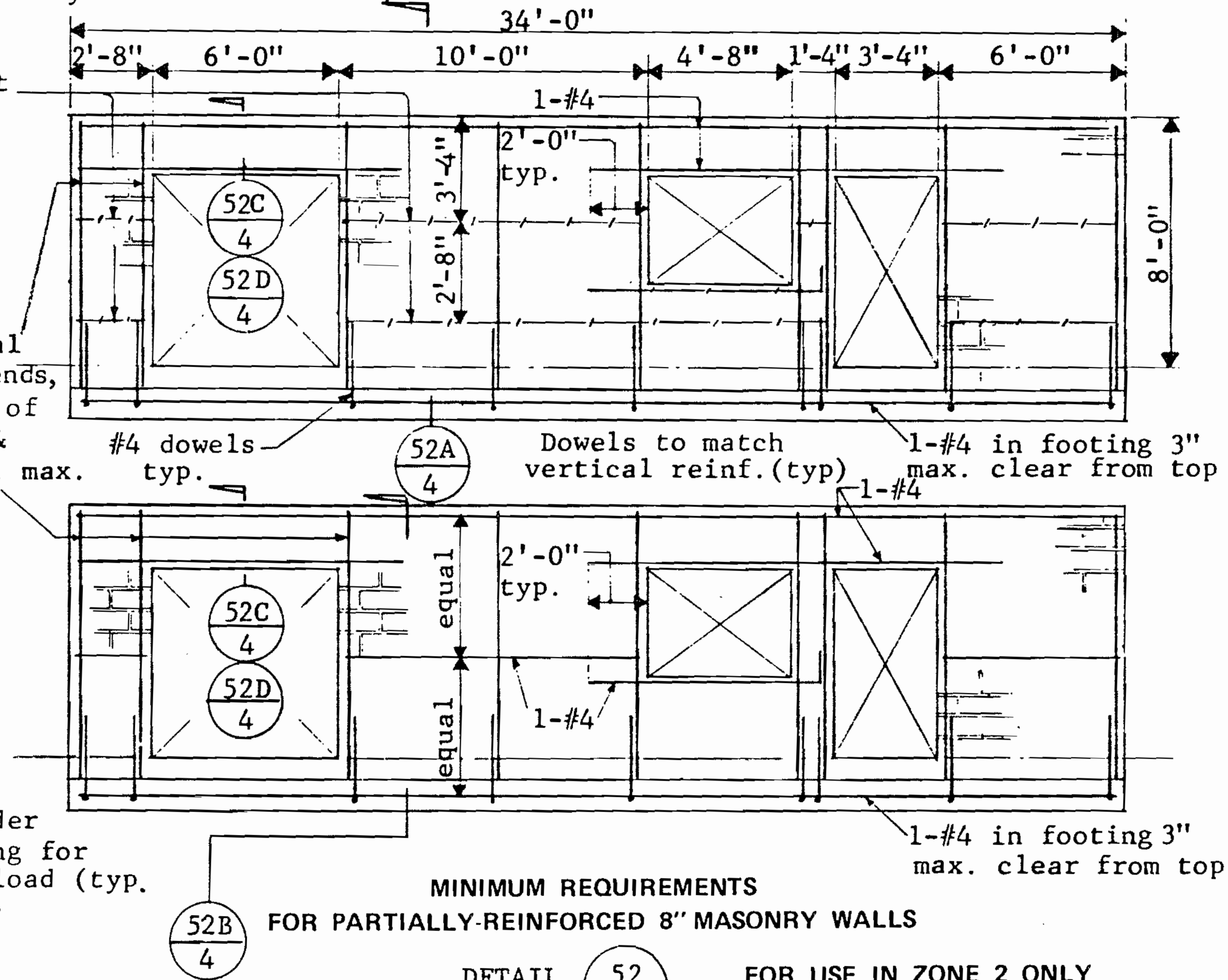
TYPICAL REINFORCING OF BRICK MASONRY LINTELS

Note: See other publications for required reinforcing of lintels to support vertical loads



SECTION 51A/4 SECTION 51B/4

Note: Equivalent bond beam and lintel reinforcement may be used in lieu of that shown. All reinforcement shown in accordance with UBC requirements for partially reinforced masonry.

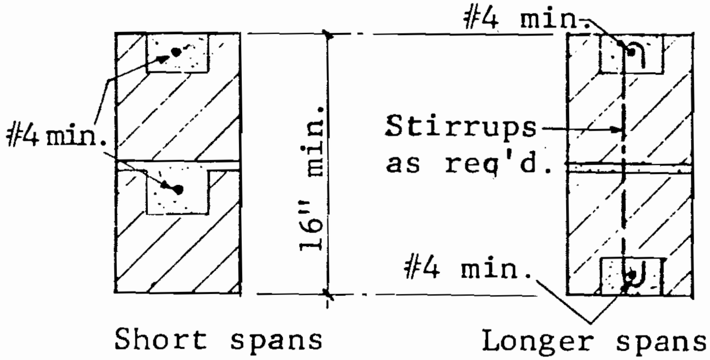


Note:
Check header reinforcing for vertical load (typ. all cases)

MINIMUM REQUIREMENTS
FOR PARTIALLY-REINFORCED 8" MASONRY WALLS

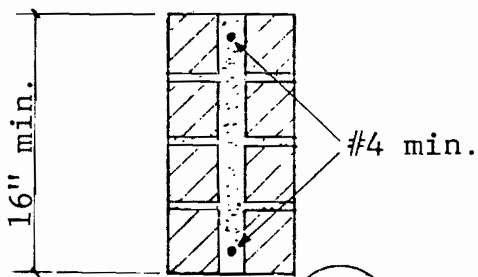
DETAIL 52/4

FOR USE IN ZONE 2 ONLY



DETAIL 52C/4

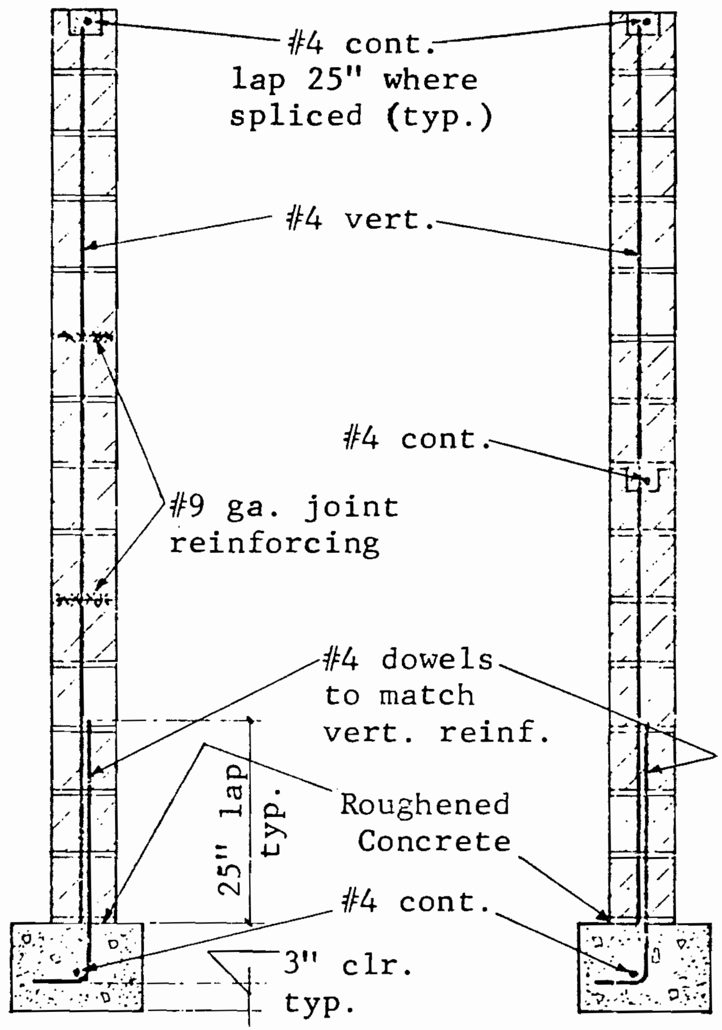
TYPICAL REINFORCING
OF CONCRETE BLOCK LINTELS



DETAIL 52D/4

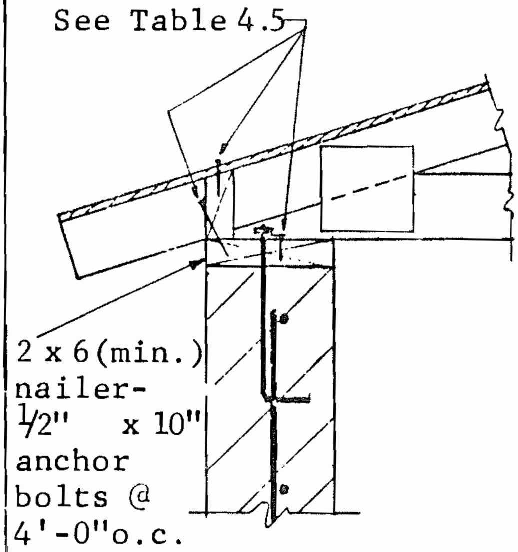
TYPICAL REINFORCING
OF BRICK MASONRY LINTELS

Note: See other publications for required reinforcing of lintels to support vertical loads

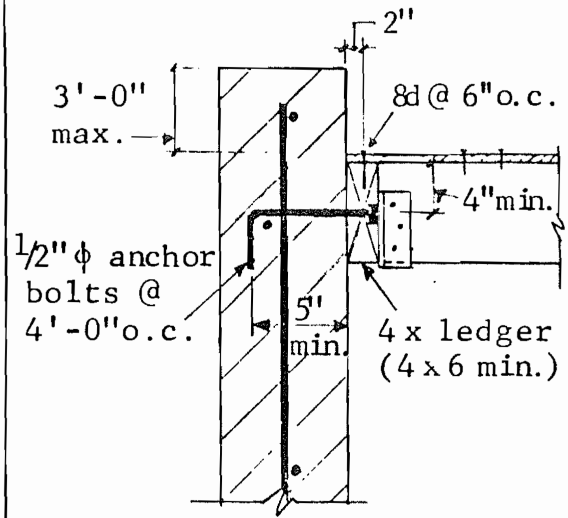


SECTION 52A/4

SECTION 52B/4

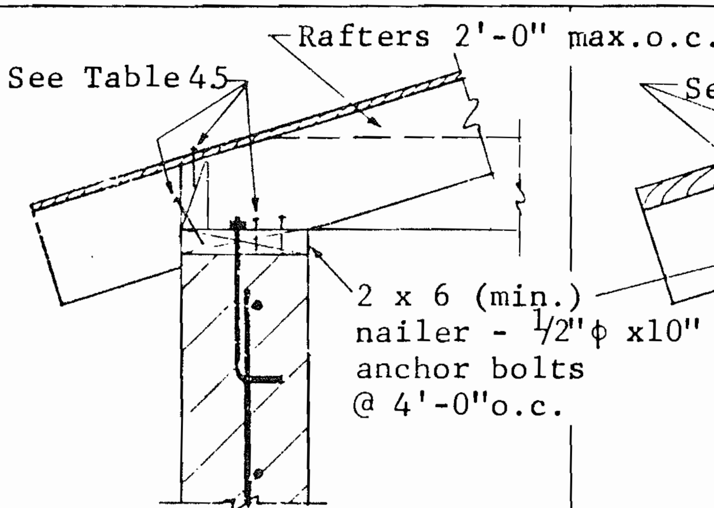


DETAIL 53A/4

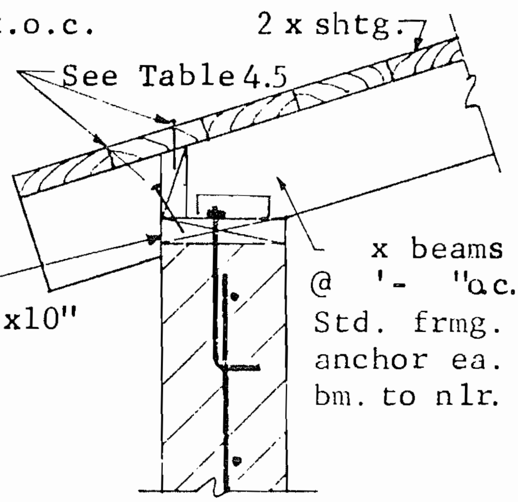


DETAIL 53B/4

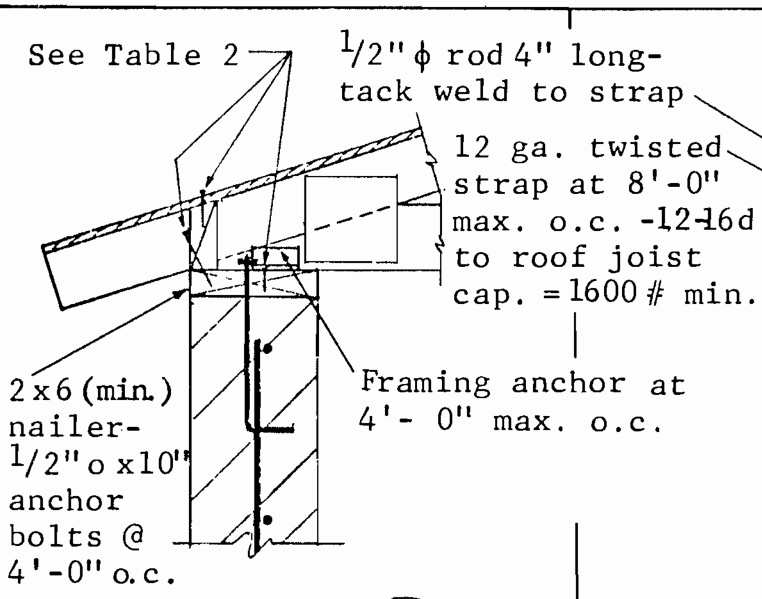
EXTERIOR MASONRY BEARING WALL – ZONE 2



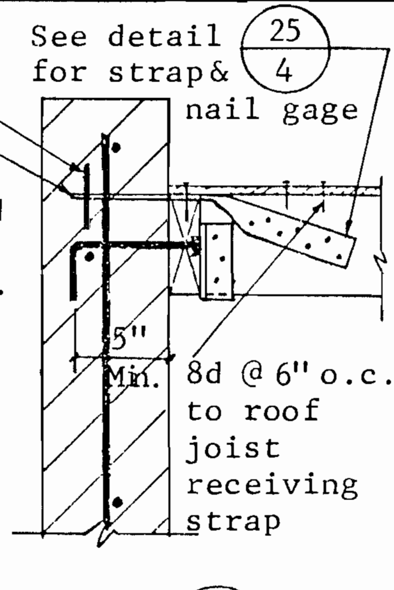
DETAIL 53C/4



DETAIL 53D/4

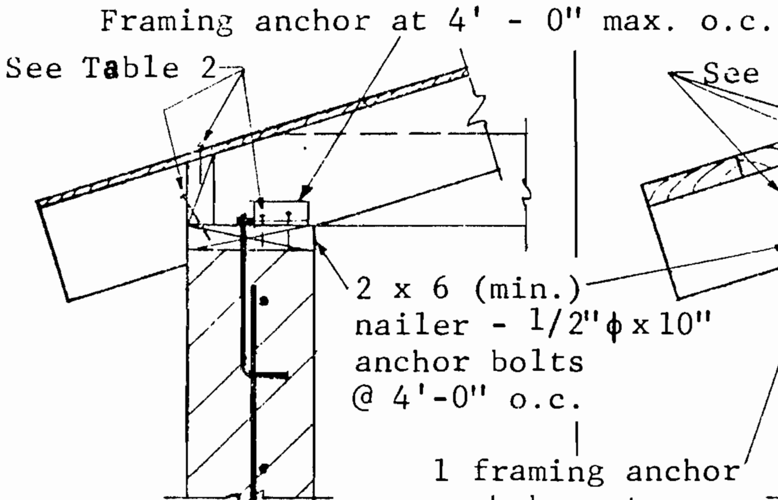


DETAIL 53A/4

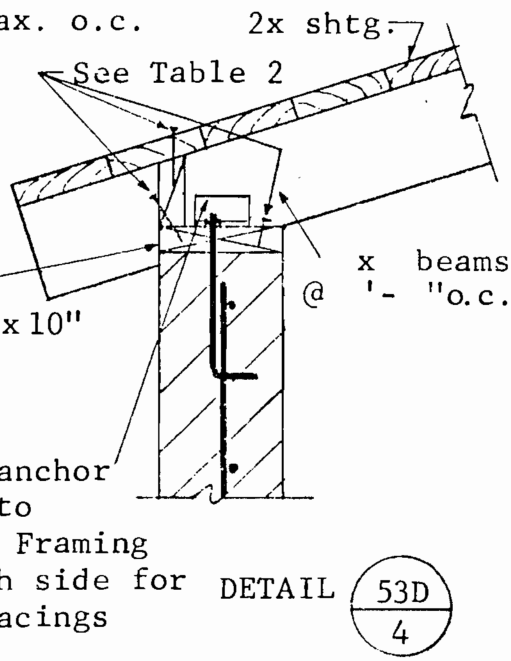


DETAIL 53B/4

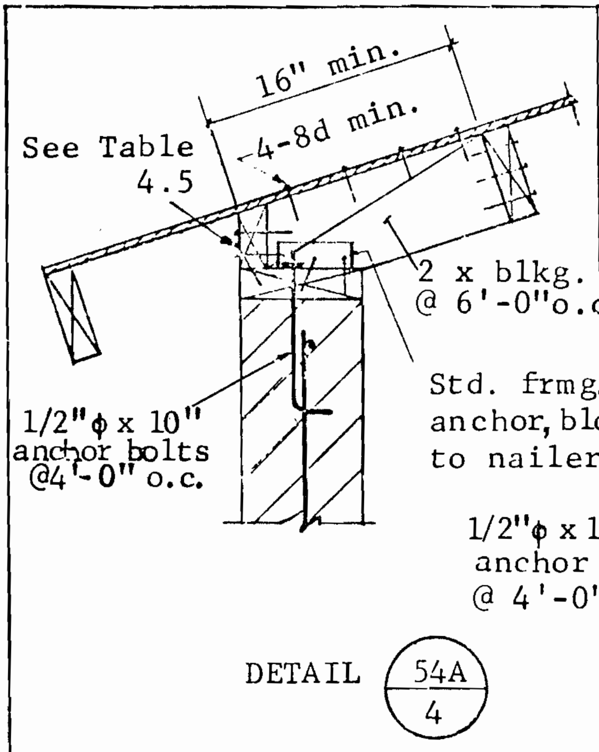
EXTERIOR MASONRY BEARING WALL – ZONE 3

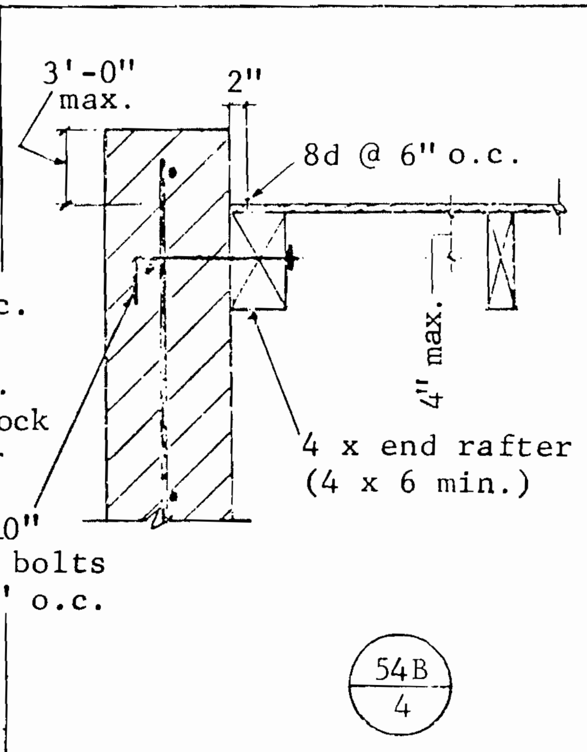


DETAIL 53C/4

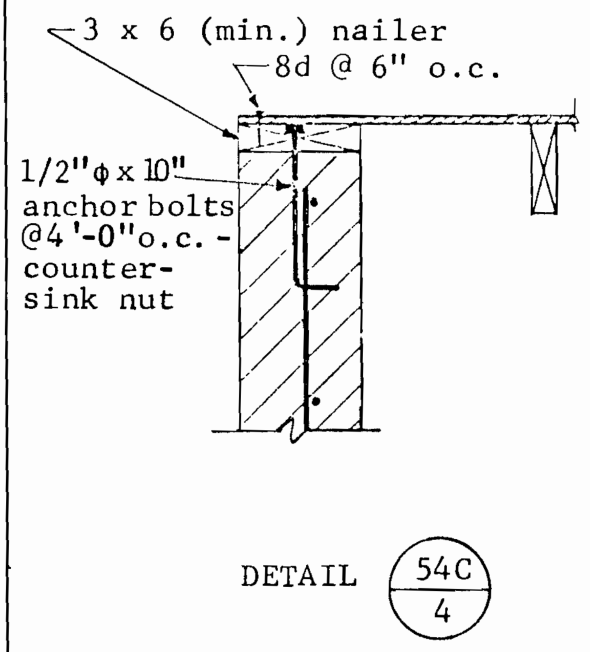


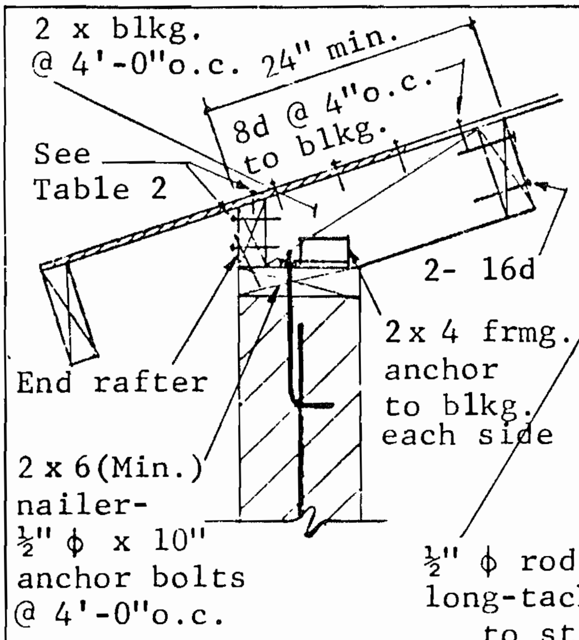
DETAIL 53D/4



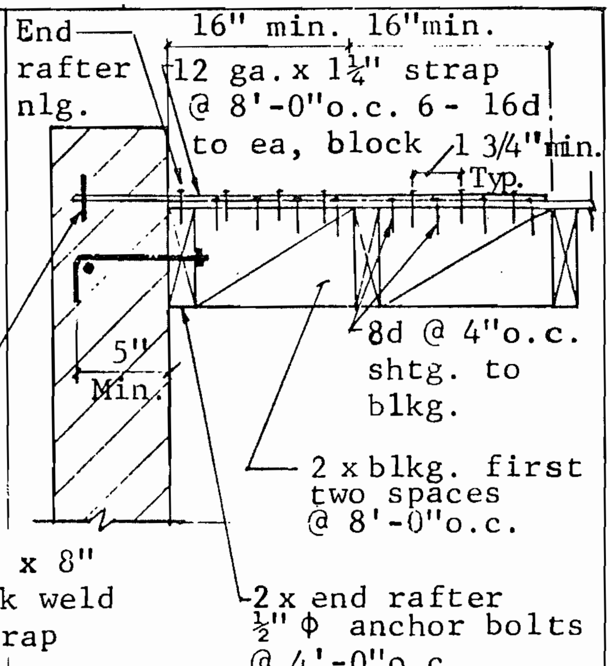


EXTERIOR MASONRY END WALL – ZONE 2



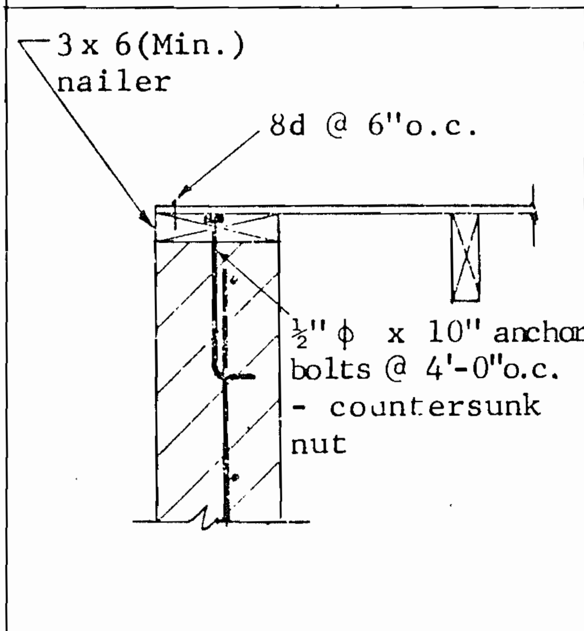


DETAIL 54A/4

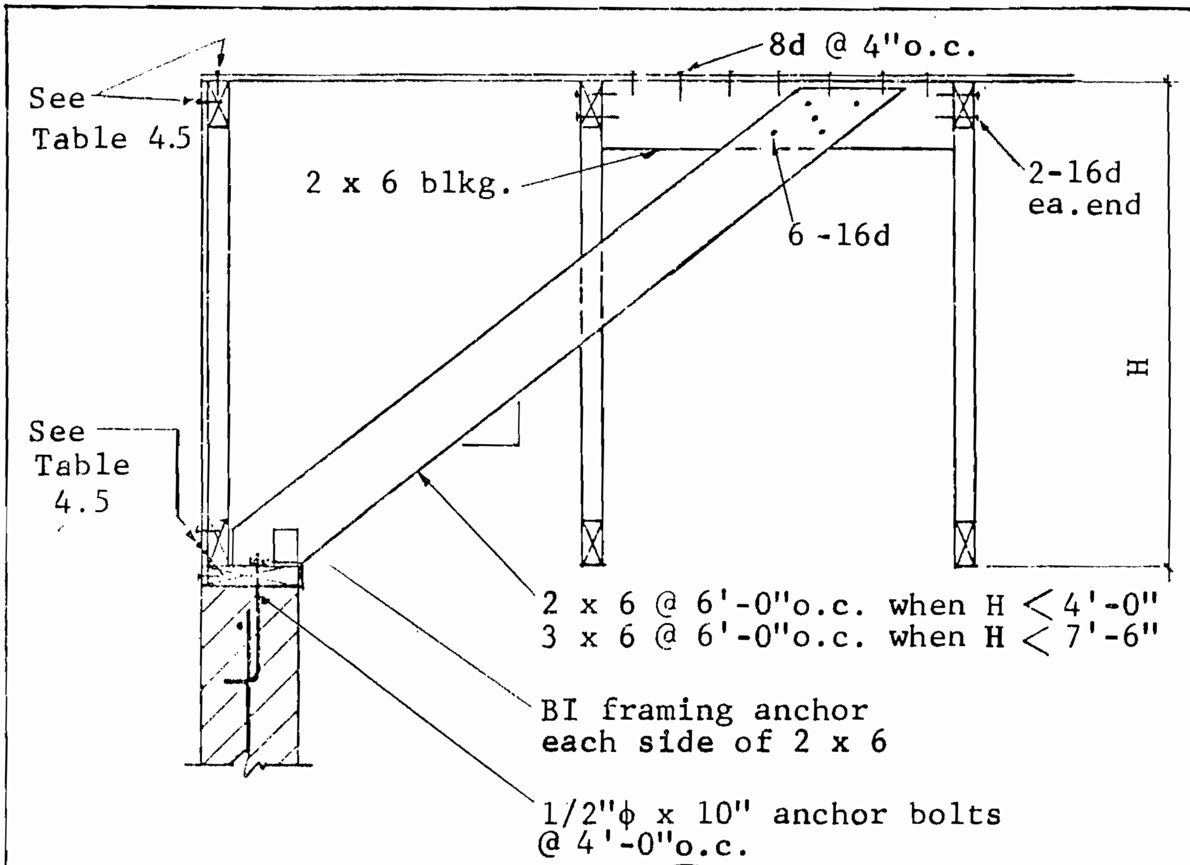


DETAIL 54B/4

EXTERIOR MASONRY END WALL – ZONE 3



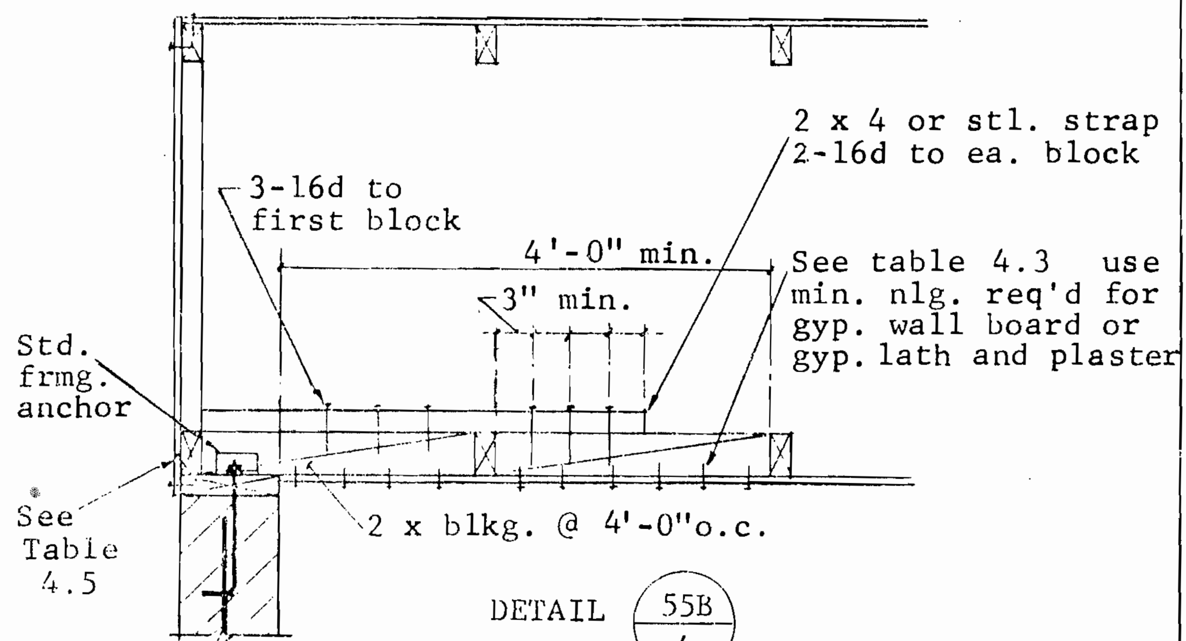
DETAIL 54C/4



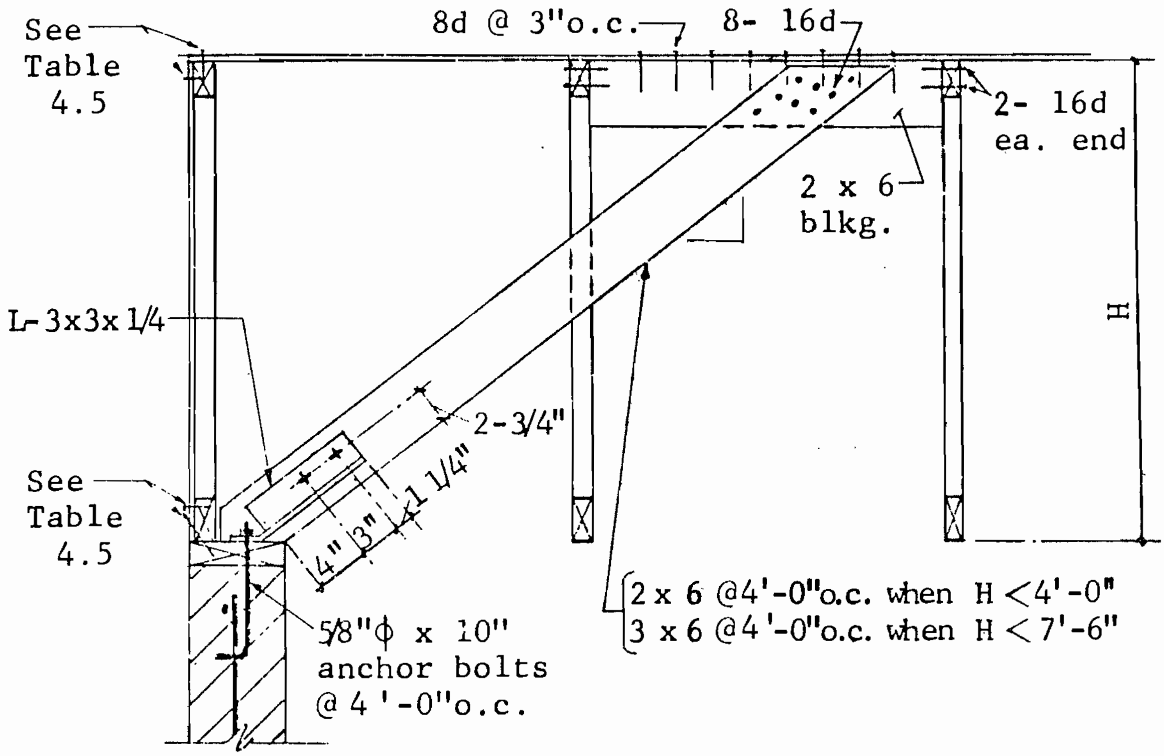
EXTERIOR MASONRY
END WALL – ZONE 2

DETAIL 55A/4

Masonry wall to ceiling line with wood framing above



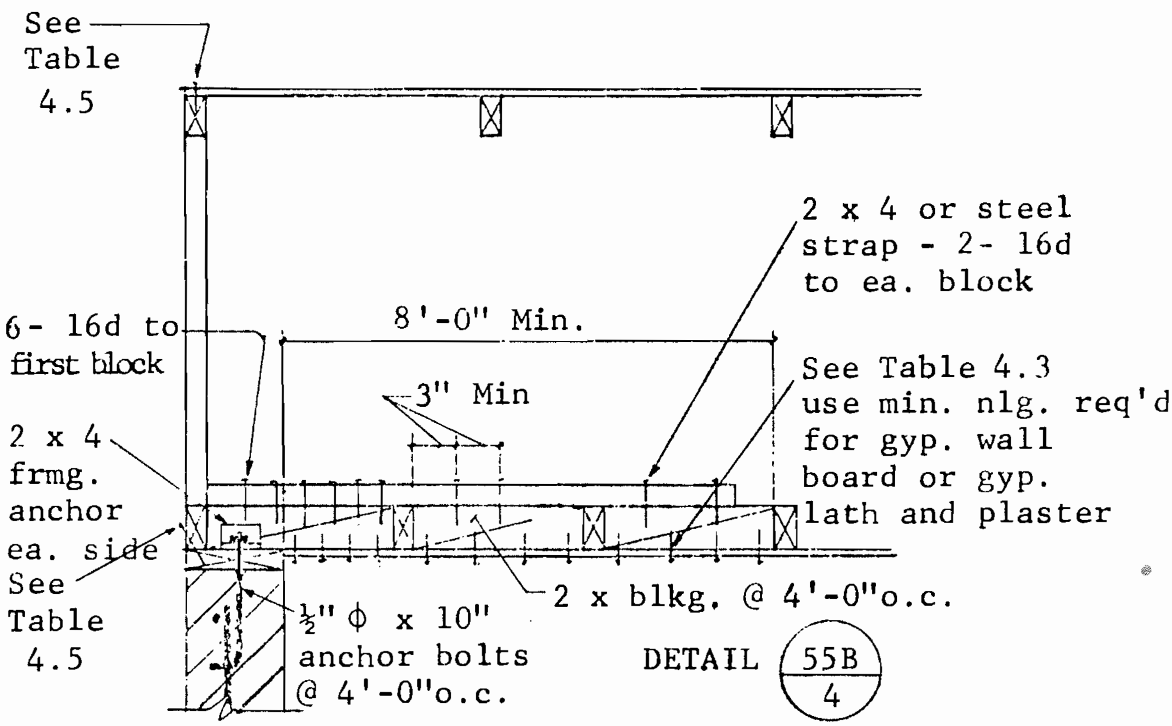
DETAIL 55B/4



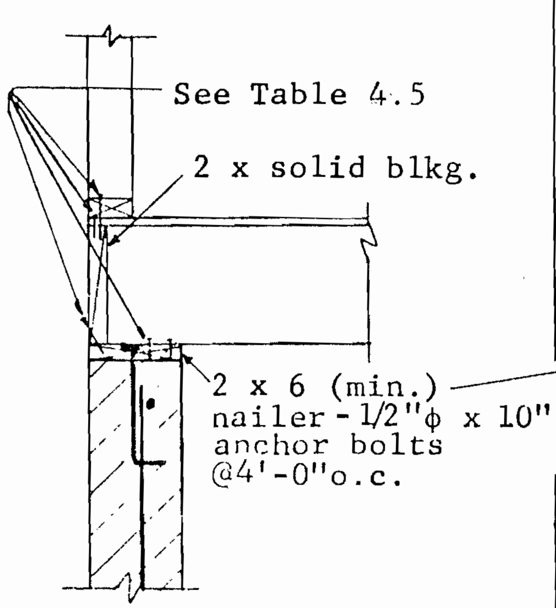
EXTERIOR MASONRY END WALL – ZONE 3

DETAIL 55A/4

Masonry wall to ceiling line with wood framing above

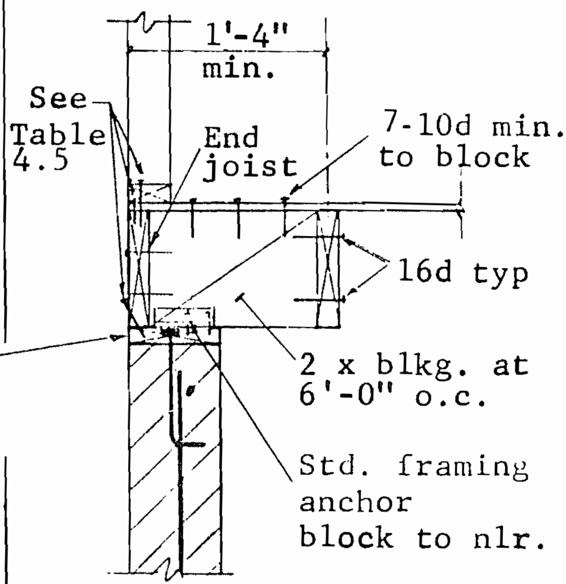


DETAIL 55B/4



DETAIL 56/4

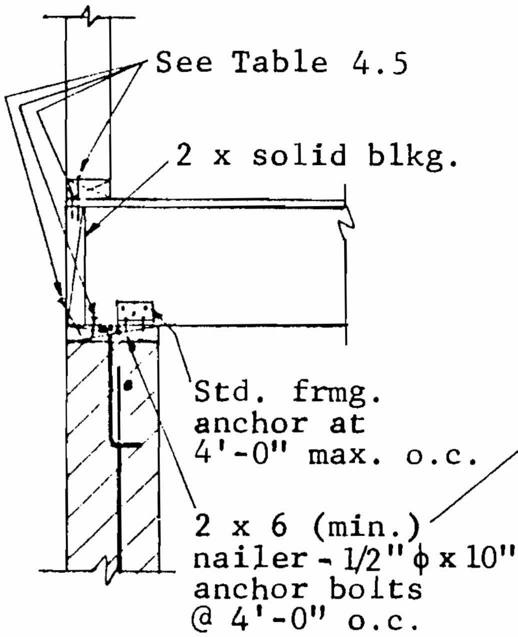
BEARING WALL



DETAIL 57/4

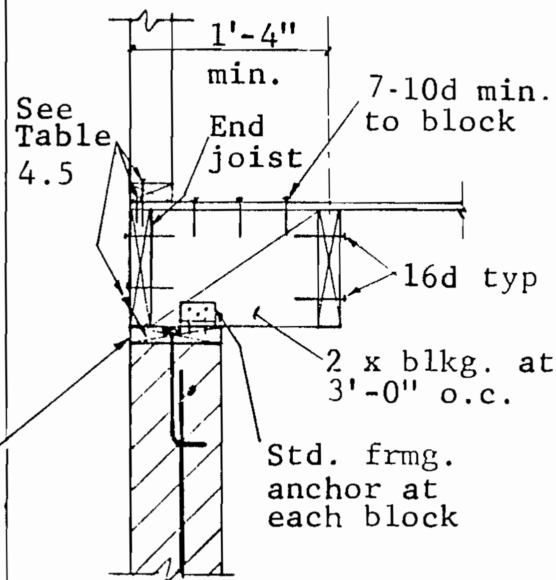
NON-BEARING WALL

EXTERIOR MASONRY WALLS SUPPORTING SECOND FLOOR – ZONE 2



DETAIL 56/4

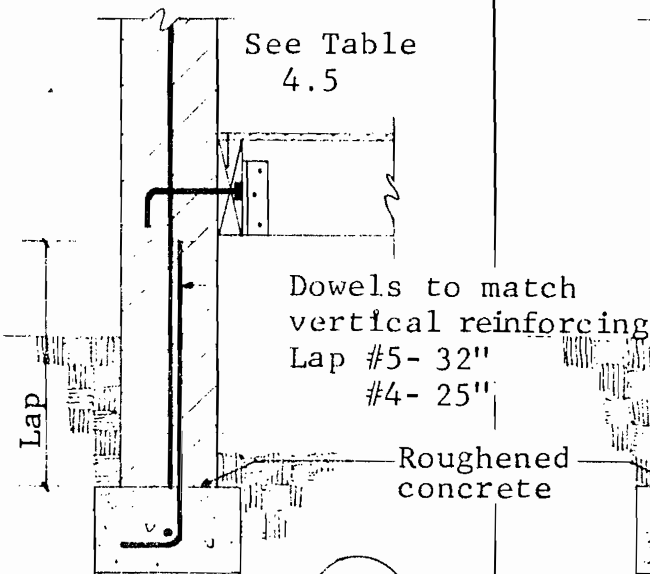
BEARING WALL



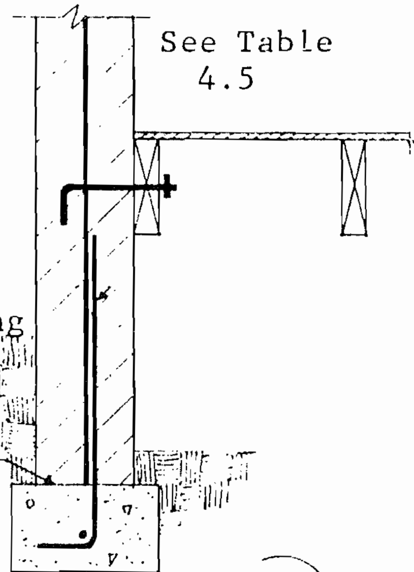
DETAIL 57/4

NON-BEARING WALL

EXTERIOR MASONRY WALLS SUPPORTING SECOND FLOOR – ZONE 3

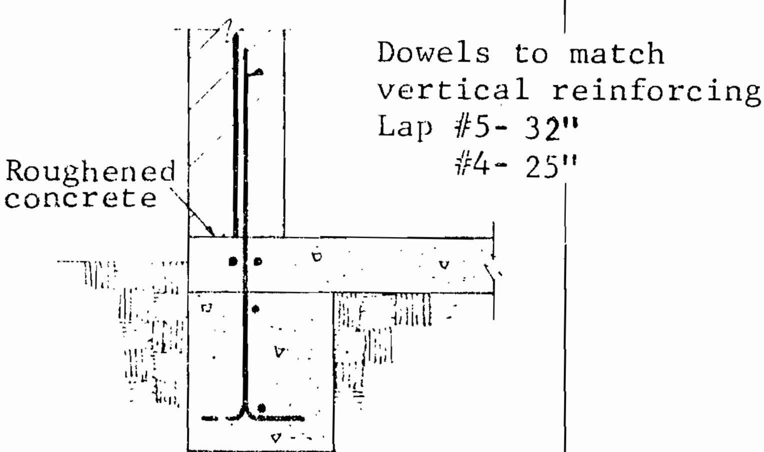


DETAIL 58/4



DETAIL 59/4

TYPICAL FOOTINGS AT EXTERIOR MASONRY WALLS



DETAIL 60/4

TABLE 4.3

ALLOWABLE SHEAR IN POUNDS PER FOOT FOR SINGLE-MATERIAL SHEAR WALL ASSEMBLIES WITH 2 x 4 OR LARGER STUDS AT 16" o.c.

MATERIAL	THICKNESS	WALL CONSTRUCTION	MAXIMUM NAIL SPACING(1) (INCHES)	SHEAR VALUE	MINIMUM NAIL SIZE
1" x 6" Diagonal Sheathing	3/4"	Unblocked	2 per board interior 3 per board boundary	300	8d common
1" x 8" Diagonal Sheathing	3/4"	Unblocked	3 per board interior 4 per board boundary	300	8d common
Gypsum Sheathing Board	1/2" x 2' x 8'	Unblocked	4	75	No. 11 gage 1-3/4" long, 7/16" head, diamond-point, galv.
	1/2" x 4'	Blocked	4	175	
Gypsum Wallboard	1/2"	Unblocked	7	100	5d cooler or 5d (.086" wire dia.) x 1-5/8" long, 9/32" concave head gypsum board nail or GWB-54 (.098 gage, 1-1/4" long, 1/4" head, annular ring)
			4	125	
		Blocked	7	125	
			4	150	
	5/8"	Blocked	4	175	6d (corresp. to above)
Fiberboard ASTM Designation C208 or D2277	7/16" x 4' x 8'	Applied vertically. Blocked	3" at all edges 6" at interior of sheets	125	No. 11 gage galv. roofing nail 1-1/2" long, 7/16" head
	25/32" x 4' x 8'			175	No. 11 gage galv. roofing nail 1-3/4" long, 7/16" head
	1/2" x 4' x 8' Nailbase			175	No. 11 gage galv. roofing nail 1-1/2" long, 7/16" head
Stucco Woven or Welded Wire Lath and Portland Cement Plaster	7/8"	Unblocked	6	180	No. 11 gage 1-1/2" long with 7/16" diameter head nail or No. 16 gage staples having 7/8" long legs
Gypsum lath plain or perforated	3/8" Lath and 1/2" Plaster	Unblocked	5	100	No. 13 gage 1-1/8" long, 19/64" head, plasterboard blued nail
Hardboard(2)	7/16"(3)	Applied vertically. Blocked	4" at all edges 8" at interior of sheets	230 shiplap vert. joints	6d box, galv.
				300 butted vert. joints	

(1) Applies to nailing at all studs, top and bottom plates, and blocking.
(2) For all hardboards meeting requirements of Appendix D of HUD Minimum Property Standards and Voluntary Product Standard PS60-73.
(3) May be notched to 1/4" nominal thickness for architectural effects.

Recommended shear in pounds per foot for plywood shear walls

for wind or seismic loading (a)

(TABLE 4.4)

Plywood Grade	Minimum Nominal Plywood Thickness (inches)	Minimum Nail Penetration in Framing (inches)	Plywood Applied Direct to Framing: Nail Size (Common or Galvanized Box)	Nail Spacing at Plywood Panel Edges (inches): 6	4	2½	2	Plywood Applied over ½" Gypsum Sheathing: Nail Size (Common or Galvanized Box)	Nail Spacing at Plywood Panel Edges (inches): 6	4	2½	2
STRUCTURAL I C-D INT-DFPA or STRUCTURAL I EXT-DFPA	5/16 or ¼(b)	1¼	6d	200	300	450	510	8d	200	300	450	510
	⅜	1½	8d	280	430	640	730	10d	280	430	640(d)	730(d)
	½	1⅝	10d	340	510	770(d)	870(d)	—	—	—	—	—
C-C EXT-DFPA, STRUCTURAL II INT-DFPA, STANDARD C-D INT-DFPA, DFPA Panel Siding, and other DFPA grades(c)	5/16 or ¼(b)(e)	1¼	6d	180	270	400	450	8d	180	270	400	450
	⅜(f)	1½	8d	260	380	570	640	10d	260	380	570(d)	640(d)
	½	1⅝	10d	310	460	690(d)	770(d)	—	—	—	—	—
			Nail Size (Galvanized Casing)					Nail Size (Galvanized Casing)				
DFPA Plywood Panel Siding applied with casing nails(c)	5/16(b)(e)	1¼	6d	140	210	320	360	8d	140	210	320	360
	⅜(f)	1½	8d	160	240	360	410	10d	160	240	360	410

(a) All panel edges backed with 2-inch nominal or wider framing. Plywood installed either horizontally or vertically. Space nails at 12 in. on center along intermediate framing members. (b) Minimum recommended when applied direct to framing as exterior siding is ⅜" or 303-16 o.c. (c) Except Group 5 species. (d) Reduce tabulated shears 10% when boundary members provide less than 3-inch-nominal nailing surface. (e) Applies also to 303-16 o.c. siding, ⅜" and thinner. (f) Applies also to 303-24 o.c. siding, and to 303-16 o.c. siding thicker than ⅜".

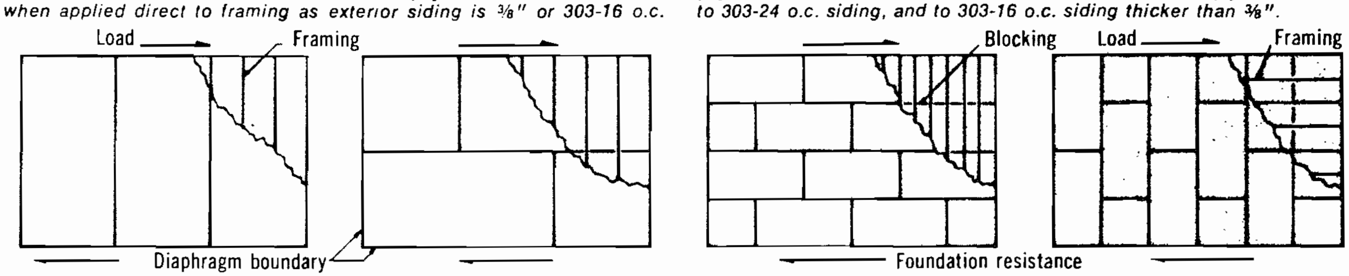


TABLE 4.5

NAILING SCHEDULE – ZONE 2

Connection	Nailing[1]
Sheathing	
Plywood – Roof – 3/8" to 3/4"	
All edges of sheets	8d @ 6" o.c.[2]
Interior bearings (field)	8d @ 12" o.c.[2]
Plywood – Floor – 1/2" to 3/4"	
All edges of sheets	10d @ 6" o.c.[2]
Interior bearings (field)	10d @ 10" o.c.[2]
Straight Sheathing	
1 x ___ sheathing board parallel to wall and over blocking	8d @ 8" o.c.
2 x ___ sheathing board parallel to wall and over blocking	16d @ 12" o.c.
1 x 6 or less – per bearing	2 – 8d
1 x 8 and 1 x 10 – per bearing	3 – 8d
1 x 12 – per bearing	4 – 8d
2 x 10 or less – per bearing	2 – 16d
Roof and Floor Framing	
Rafters, ceiling joists or 2nd floor joists to top plate – toenail	3 – 8d
Ceiling joists to parallel rafters – face nail	3 – 16d
Solid blocking at roof to top plate	
Up to 36" long – toenail – per block	1 – 16d
Up to 48" long – toenail – per block	2 – 16d
Over 48" long	16d @ 16" o.c.
Partial blocking at roof to top plate	
2 x ___ block x 21" min. long @ 10'-0" o.c. – toenail – per block	6 – 16d
2 x ___ block x 13" min. long @ 8'-0" o.c. – toenail – per block	4 – 16d
*Blocking at 2nd floor or 1st floor of 1-story	
Up to 16" long – toenail – per block	1 – 16d
Up to 24" long – toenail – per block	2 – 16d
Over 24" long – toenail	16d @ 8" o.c.
*Blocking at 1st floor of 2-story	
Up to 16" long – toenail – per block	1 – 16d
Up to 24" long – toenail – per block	2 – 16d
Up to 48" long – toenail	16d @ 8" o.c.
End rafter to top plate – toenail	16d @ 16" o.c.
Bottom chord of end truss to top plate – toenail	16d @ 12" o.c.
*End floor joist to top plate – toenail	16d @ 6" o.c., or framing anchor @ 4'-0" o.c.
Wall Framing	
Double top plates – facenail	16d @ 16" o.c.
Double top plates – laps and intersections – facenail	2 – 16d
Double top plates – min. total nails between splices	9 – 16d
Top plate to stud – endnail	2 – 16d
Stud to sole plate – toenail	4 – 8d
Doubled studs and corner studs – facenail	16d @ 24" o.c.
*Sole plate to joist or blocking	
One story or at 2nd floor of two-story – facenail	16d @ 16" o.c.
First floor of 2-story – facenail	16d @ 12" o.c.

*When exterior finish material is continuous past 2nd floor framing or continuous past 1st floor framing and attached to mudsill, all nailing may be 1/2 of that indicated except that sole plate nailing may not be less than 16d @ 16" o.c.

(1) Common or box nails may be used except where otherwise stated.

(2) Common nails only.

TABLE 4.5

NAILING SCHEDULE – ZONE 3

Connection	Nailing[1]
Sheathing	
Plywood – Roof – 3/8" to 3/4"	
All edges of sheets	8d @ 6" o.c.[2]
Interior bearings (field)	8d @ 12" o.c.[2]
Plywood – Floor – 1/2" to 3/4"	
All edges of sheets	10d @ 6" o.c.[2]
Interior bearings (field)	10d @ 10" o.c.[2]
Straight Sheathing	
1 x ___ sheathing board parallel to wall and over blocking	8d @ 8" o.c.
2 x ___ sheathing board parallel to wall and over blocking	16d @ 12" o.c.
1 x 6 or less – per bearing	2 – 8d
1 x 8 and 1 x 10 – per bearing	3 – 8d
1 x 12 – per bearing	4 – 8d
2 x 10 or less – per bearing	2 – 16d
Roof and Floor Framing	
Rafters, ceiling joists or 2nd floor joists to top plate – toenail	3 – 8d
Ceiling joists to parallel rafters – facenail	3 – 16d
Blocking at roof to top plate	
Up to 24" long – toenail – per block	2 – 16d
Up to 48" long – toenail	16d @ 8" o.c.
Over 48" long – toenail	16d @ 6" o.c.
*Blocking at 2nd floor or 1st floor of 1-story	
Up to 16" long – toenail – per block	3 – 16d
Up to 24" long – toenail – per block	5 – 16d
Over 24" long – toenail	16d @ 4" o.c.
*Blocking at 1st floor of 2-story	
Up to 16" long – toenail – per block	4 – 16d
Up to 24" long – toenail – per block	6 – 16d
Up to 48" long – toenail (staggered)	16d @ 3" o.c., or standard framing anchor @ 2'-4" o.c.
End rafter to top plate – toenail	16d @ 12" o.c.
Bottom chord of end truss to top plate – toenail	16d @ 6" o.c.
*End floor joist to top plate – toenail (staggered)	16d @ 3" o.c., or standard framing anchor @ 2'-0" o.c.

Nailing listed below may be substituted for the nailing shown above for items indicated if house meets following criteria:

Exterior finish: Any but stucco; Interior finish: Any but lath and plaster;
Roofing: 6 psf maximum; Veneer: 4"-thick maximum, two perpendicular sides max.

Connection	Nailing
Blocking at roof to top plate	
Up to 24" long – toenail – per block	1 – 16d
Up to 48" long – toenail	16d @ 10" o.c.
Over 48" long – toenail	16d @ 8" o.c.
*Blocking at 2nd floor or 1st floor of 1-story	
Up to 16" long – toenail – per block	2 – 16d
Up to 24" long – toenail – per block	4 – 16d
Over 24" long – toenail	16d @ 4½" o.c.

(Table 4.5 continued on reverse side)

TABLE 4.5 Continued

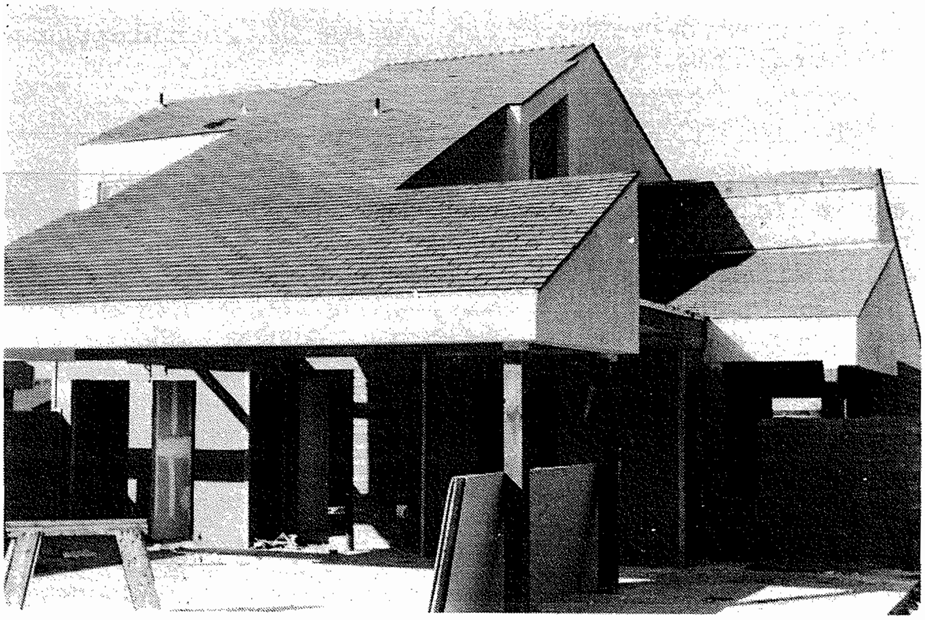
NAILING SCHEDULE – ZONE 3

Connection	Nailing(1)
*Blocking at 1st floor of 2-story	
Up to 16" long – toenail – per block	3 – 16d
Up to 24" long – toenail – per block	5 – 16d
Up to 48" long – toenail	16d @ 4" o.c.
*End floor joist to top plate – toenail	16d @ 4" o.c., or framing anchor @ 2'-6" o.c.
Wall Framing	
Double top plates – facenail	16d @ 16" o.c.
Double top plates – laps and intersections – facenail	2 – 16d
Double top plates – min. total nails between splices or intersections	9 – 16d
Top plate to stud – endnail	2 – 16d
Stud to sole plate – toenail	4 – 8d
Doubled studs – facenail	16d @ 24" o.c.
Corner studs	16d @ 24" o.c.
*Sole plate to joist or blocking	
One story or at 2nd floor of 2-story – facenail	16d @ 8" o.c.
1st floor of 2-story – facenail	16d @ 4" o.c.

*When exterior finish material is continuous past 2nd floor framing or continuous past 1st floor framing and attached to mudsill, nailing may be 1/2 of that indicated except that sole plate nailing may not be less than 16d @ 16" o.c.

(1) Common or box nails may be used except where otherwise stated.

(2) Common nails only.



PART V GUIDELINES FOR PLAN-CHECKERS AND INSPECTORS

CHAPTER V–1 PLAN-CHECKING

To insure the proper implementation of the methodology prescribed in this report, the plan-checker must become thoroughly familiar with all the methods and data presented. In addition to determining whether home designs meet the requirements for seismic resistance, the plan-checker should be knowledgable about the principles of seismic design when faced with questions that are certain to arise. Proper checking to determine whether plans meet seismic requirements consists of two steps:

- First, that calculations and design are correct and complete;
- Second, that plans include proper and appropriately-referenced detailing.

Checking Calc Forms

One of the best devices in design-checking is for the checker to prepare, in rough form, his own analysis. Check of only a few each of the basic one-story, two-story and split-level homes will provide the checker sufficient experience to determine which walls need design calculations to determine their adequacy. Since they will be using this document less frequently than HUD plan-checkers, it is probable that most designers will calculate more walls than necessary. These "extra" walls can be reviewed simply, merely to determine that they require only minimal sill bolting, shear material nailing, etc. Once the checker has determined which walls require design, he need inspect only whichever Calc Form (8 or 9) is used to determine that the necessary calculations have been made.

Lines of Shear Resistance—Tributary Areas and Material Weights

It is recommended that the checker determine independently the material weights and the tributary area for each wall to be designed. After having determined these and before going further, Calc Form 1, or 2 and 3, as applicable, should be reviewed to determine that the weights are proper, and Calc Form 6 should be checked to see that there is essential agreement with the tributary areas used. Due to differences in assumptions, some variations are to be expected. It should not be demanded, for instance, that 3.0-pound roofing weight be used when a designer has used 2.7 pounds. Differences of opinion also may arise over whether to include relatively insignificant weights such as light-weight insulation and other such materials. While adding these weights does no harm, accuracy to each tenth of a pound is not critical. Far more important, for instance, is the determination that masonry veneer loads have not been forgotten, or that proper wall loads have been selected and entered on the Calc Forms correctly.

Similarly, determination of tributary areas may vary. The tributary area plans in Chapter II—2 were prepared in detail for presentation purposes only. It is unnecessary for the designer to prepare such complete plans to accompany the calculations. It may prove helpful to require at least a rough, single-line sketch indicating lines of resistance the designer has assumed plus width and lengths of area assumed tributary to each line. Such sketches need not be to scale or fully dimensioned, and may not be needed at all for the simpler designs.

When a large difference between area determined by the checker and area submitted by the designer occurs, however, it is suggested that the checker first attempt to determine whether the manner of distribution of lateral forces assumed by the designer is acceptable though different from the checker's manner. If this does not appear to be the case, or if the checker cannot readily determine what assumptions the designer has made, a correction should be written. Even when such a variation exists, it is recommended that the checker complete the design review, preparing his own notations, no matter how sketchy, in such a way that he can discuss them with the designer or use them in the second check after the corrections have been made.

Seismic Forces

It is recommended that if there is essential agreement with his own notations, the checker use the designer's tributary areas and equivalent seismic weights to check seismic forces and shear wall design. Once areas and equivalent seismic weights have been determined, the only disagreements should result from arithmetical errors or omissions in determining seismic forces. The designer, for instance, could forget to include the seismic force from a masonry fireplace, from veneer or from other similar items; or, he could transpose numbers or enter them in a wrong column. Such errors or omissions are more likely to be caught if the checker makes his own calculations. The calculations need not be prepared on Calc Forms, nor is it necessary that every simple condition be cross-checked. For one-story homes only two or three walls, on the average, will require checking. More checking will be required for two-story and split-level homes which are much more susceptible to earthquake damage; the extra design and checking time will be the prospective homeowner's best insurance against potential major damage to his residence.

Shear Wall Design

In checking shear wall designs the checker should first determine the length of shear wall along each line of resistance being reviewed and determine the length that may be considered for overturning purposes. (The checker may wish to annotate these lengths in a small tabulation at a side of the sheet he uses for rough-checking the calculations.) He then should determine that seismic forces entered on Calc Form 8 or 9 match those determined on Calc Form 7. (When Calc Form 9 is used, a separate check is needed to determine whether wind loads have been arrived at correctly.) Next, the checker should determine the shear-per-foot and then check sill bolt spacing, adequacy of shear materials and overturning requirements using the various tables and graphs accompanying the Design Methodology.

Information on Plans

Before the Calc Forms are set aside it is recommended that the checker review the roof framing plan, floor plan, foundation plan, elevations, etc., to determine that any special requirements with regard to sill bolt spacing, special nailing of shear materials, and overturning requirements have been referenced on the plans. He should also determine whether struts are required, in accordance with the discussion in Part II and as outlined in the Design Methodology.

Checking Predesigned Details

When predesigned details are used as shown in Part IV, the checker's job is simplified. He must determine only that all required details have been included, have been referenced on the plans and are applicable. In those cases where variance from the predesigned detail is obvious and minor, the reference on the plans with the word "similar" thereafter is acceptable. Other ordinary practice in plan presentation, such as placing the word "typical" after a detail callout rather than specifying the detail at many locations on the plan, should also be accepted.

Table 4.1 will assist the checker in referring to the predesigned details. The table includes indications of applicability to houses with wood frame or masonry exterior walls or both, and notations on whether the details are normally required, rarely required or not applicable for a given type of dwelling design. Where Table 4.1 requires the use of a predesigned detail that does not adequately reflect the actual condition a substitute detail should be provided.

It is important that the checker understand that the predesigned details do not necessarily delineate all information required in a particular locale, but do include the information needed to make the detail sound for seismic resistance. For example, reinforcing steel has not been called out for retaining walls because unreinforced walls are acceptable in some areas of the nation. Substantial amounts of reinforcing are required in others, however. Using Detail 15/4 as another example, Details 15A and 15B indicate mesh in the slab, but mesh is not called out on the detail – it is a requirement in only some locations and its appearance in the detail should not be interpreted as an indication of a general requirement for slabs in all locations. With regard to that same detail, the contractor is given three choices of methods of attachment of the slab to the footing. The fact that none of the three is shown on any other detail (such as the cross-section on Detail 46B/4) is not intended to imply that no slab attachment is required, but rather that any one of the three choices on Detail 15 may be used.

The details do not refer to the nailing of finish materials and, in some cases, do not even show the finish. Table 4.3, or the pertinent information from the table, should be included with each set of plans. As referenced throughout these recommended requirements, all finish materials should be structurally nailed as indicated in the Design Methodology.

As stated elsewhere in this publication, the "pictures" shown for each detail are presented only to convey the information called out and it should not be inferred that the picture itself is the required method of construction. This applies particularly with respect to all details indicating "standard" framing methods. On the other hand, the "picture" is most important in the special details such as the Special Garage Front Wall Detail.

Checking Revised Details

Although the predesigned details anticipate most construction conditions in single-family dwellings, it will not be at all unusual to encounter designs for which some do not exactly apply. In such instances it is important for the checker to require a cross-section or other detail which provides an acceptable modification to the detail in question achieving the intent of the Design Methodology. Two examples:

(1) In some designs the first-floor joists are cantilevered over a continuous footing. Detail 14A, though covering a quite similar condition, does not correctly represent the situation. The designer should present a section indicating the cantilevered floor joists and showing blocking under the exterior wall as well as at the foundation wall in a manner similar to that shown on Detail 4A/4.

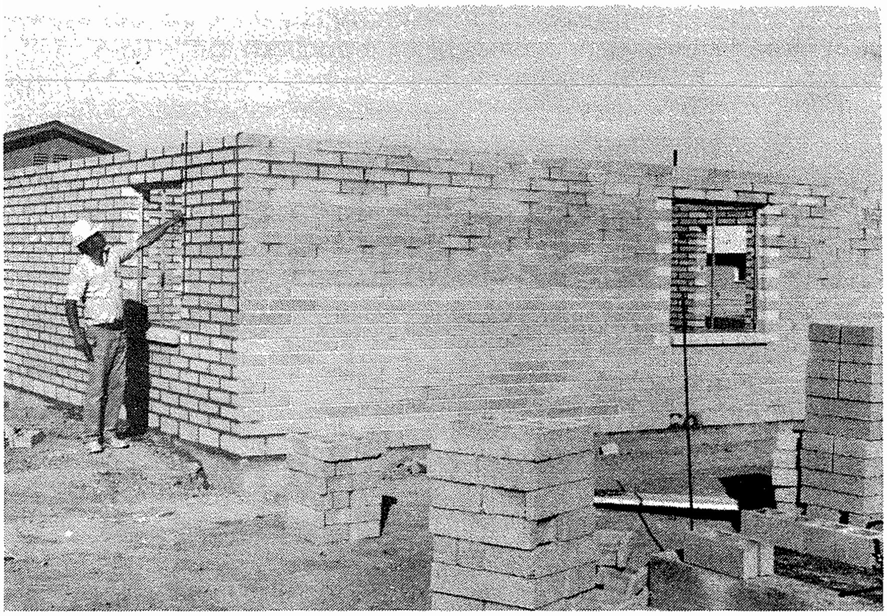
(2) Another example involves the infrequent circumstance when a strut is required. The predesigned details show struts as if they are roof members, but the slope of the strut could be level such as at a floor, or even downward from the wall rather than upward as shown. The important points on these details are whether a single or double member is required and the connection to the top plates. Using the detail as presented for the installation of a strut in a second floor would probably be confusing to the contractor. The strut details can be used in this case only to allow the designer to ascertain the connection and number of members required to develop a proper strut; it is highly preferable that the condition be drawn as it exists rather than referring to it as "similar" to one of the existing details.

Provision for Engineering

While the Design Methodology is intended to provide for improved seismic-resistant design of houses without the necessity of structural engineering, conditions may arise for which the Methodology offers no clearcut solution. In such cases it is recommended that a correction be written requesting that engineering be provided.

Local Acceptable Standards

Undoubtedly, special localized conditions or construction procedures will merit consideration of revision of some of the predesigned details to meet local practices. In those instances, Local Acceptable Standards should be developed and promulgated and it is recommended that such "Standards" provide acceptable substitute details. Should the special circumstances be general rather than localized, revision of the design recommendations presented herein should be made.



CHAPTER V–2 INSPECTION

The development of a shear path from the roof or floor sheathing through the blocking to the top plates and thence to the shear-resistant (finish) material down the wall and from the finish material to the plate and thence to the foundation is the sole purpose of many of the predesigned details presented in Part IV. To develop this shear path the nailing between framing members has been set forth in the Nailing Schedule and, in most cases, will be found to be more restrictive than previously required. If the inspector will think of this shear path in the same manner as he automatically thinks of a path required to provide full support of vertical loads, the understanding and interpretation of the details will be made much simpler.

Although many of the details reflect procedures commonly used in home construction, each detail includes some required change or changes in standard procedure. In many cases only the amount of nailing required to connect one framing member to another is changed, but when several changes are instituted, the job of the inspector becomes a large one. In addition to the normal inspection proçedures, the inspector may find himself in the position of having to acquaint contractors and subcontractors with many of the tables and new details presented. It is essential that the inspector read Chapter II–1 to develop an understanding of the requirements and details. Technical information useful to the inspector is found in Part II, Part IV and Chapter V–1. Finally, the Design Methodology in Part III contains new standards for the construction of fireplaces, the attachment of veneer, and the tying down of mechanical equipment. It is recommended that the inspector acquaint himself with all this information rather than rely on the details alone.

It is not the intent to discuss items that may be found elsewhere herein but rather to acquaint the inspector with the types of problems to expect in the field. These can be divided into two categories: Common errors encountered in locales where similar construction is already required; and, those problems likely to be created by the impact of a relatively sizable number of individual new requirements being introduced to the housing industry at one time.

Common Field Errors

Most inspectors keep a mental file of items most frequently subject to improper construction. As for seismic resistance, three items stand out as the most troublesome:

- Nailing of finish material to sill or sole plates.
- Placement of sill bolts.
- Installation of hold-down anchors.

Nailing of Finish Material. The nailing specified in Table 4.3 for the application of finish materials may differ from existing local requirements. Although finish materials are applied correctly to all other framing, inadequate connection to the sill or sole plates is a commonly observed construction flaw. Since the chain is only as strong as its weakest link, it is essential that field inspectors stress the requirements for the nailing to these plates. When the finish material extends to the mudsill, proper nailing to the sill should always be provided. When the finish stops at the sole plate above wood floor framing, the required nailing of the sole plate is heavier and the finish must be properly attached to the sole plate.

It is also important to determine that the minimum size of sheet-type materials as specified in the Design Methodology is met. All shear-resisting materials that come in sheets should be of 2'-0" x 4'-0" minimum dimensions. Where odd dimensions (such as 21'-0") would normally allow use of 4'-0" sheets, plus a single sheet narrower than 2'-0" in width, one of the 4'-0" sheets should be cut back to insure a greater-than-2'-0" width for the last sheet. The one exception to this rule: The width between openings may be less than 2'-0" but, in this instance, the shear material should be in one piece spread out to a minimum dimension of 2'-0" above and/or below the opening.

Placement of Sill Bolts. Figures 5.1, 5.2 and 5.3 demonstrate common errors in the use of sill bolts: Placement too close to the edge of the mudsill; lack of bolts where sills abut; and, notching around the bolts. It is recommended that the inspector require installation of concrete stud anchors to replace missing or misplaced sill bolts.

Hold-down Anchors. When hold-down anchors are installed, it is most important that the finish material be nailed to the same member to which the hold-down is attached. Angle hold-downs (Detail 34/4) are bolted through both studs, which is no problem when the hold-down is located at the end of a wall. When the angle hold-down is adjacent to a door or window, however, the edge of the finish material may not occur at the doubled studs. In such cases, it is important that the inspector require the specified edge-nailing to these studs. The nailing required between doubled studs is sufficient to transfer the loads developed by

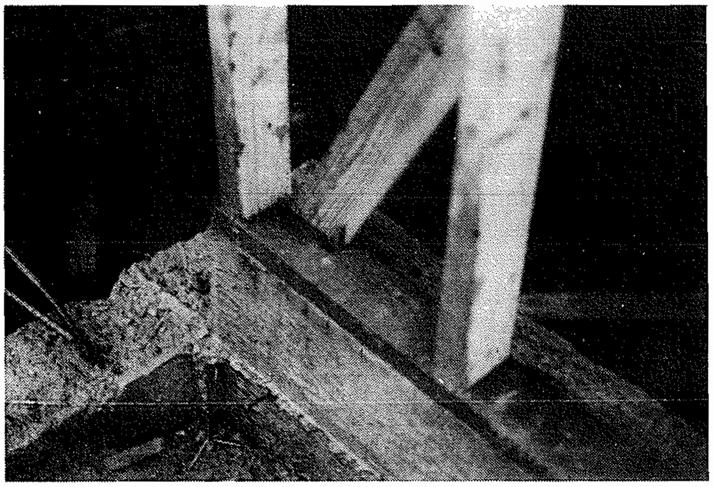


FIGURE 5.1 Sill bolts placed too close to the edge of the mudsill.

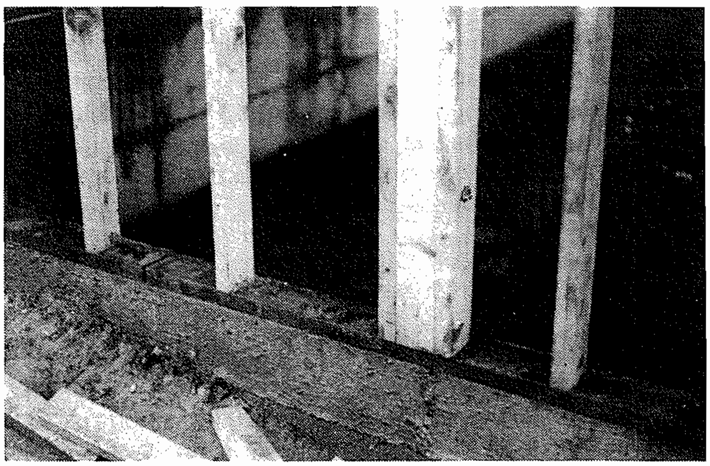


FIGURE 5.2 No bolts provided at the point where the sills abutted.

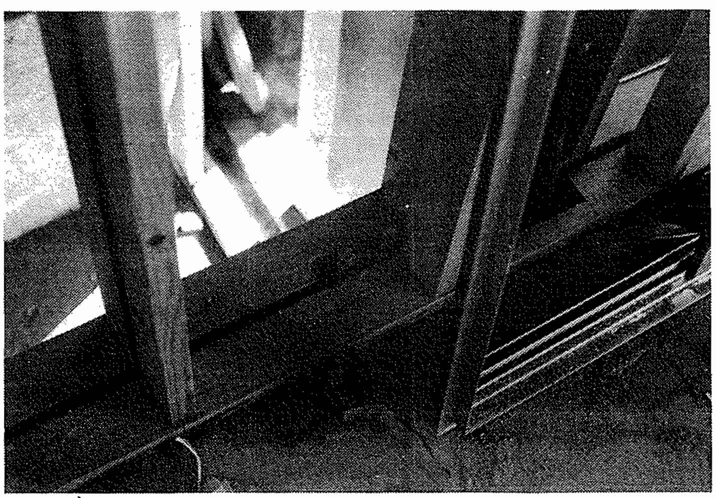


FIGURE 5.3 The wood plate was neatly notched around the bolt.

framing anchor hold-downs and for this reason, the edge nailing may be applied, in this instance, to either stud. For strap hold-downs, however, the finish and the hold-down must be nailed to the same stud.

It is anticipated that the requirements for hold-down anchors, not generally used in dwelling construction, will create the largest initial problems in the industry because of the trouble and expense involved in installing a misplaced or missing anchor in existing concrete. In some cases this may be accomplished by using concrete stud anchors attached to a blocking "bridge" spanning between the end studs and the first interior stud of the wall. In such instances the shear-resisting material must be edge-nailed to both studs. For other conditions, it may be more economical to place stud anchors into the side of the footing and extend a strap downward from the studs to the anchors. The metal straps installed in this manner must be encased in additional concrete when below ground level. In either case, the installation of the corrective hold-down must be in accordance with a detail provided by an engineer. It should be evident that to avoid these difficulties, the building contractor should be made fully aware of the importance of installing hold-down devices properly. Where practical, the use of templates for locating the hold-down is encouraged.

Other Field Problems. Particular designs often create construction conditions that are not entirely typical and just as often result in construction that is not seismically sound. Such conditions will be more apparent to the inspector who is aware of the general principles of seismic resistance as set forth in Chapter II—1. Figures 5.4 through 5.6 illustrate a few of such conditions which may not always be apparent to the plan-checker, thus increasing all the more the importance of the inspector's understanding of seismic-resistant construction methods.

In the house shown in Figure 5.4, the roof trusses have a heel height greater than normal with blocking installed between the top chords of the trusses. Sheet-type finish material probably would stop at the top plate with additional finish material installed from that point to the underside of the rafters. If this were done, the shear-resisting material above the top plates would be smaller than the stipulated 2'-0" x 4'-0" minimum size and there would also be no transfer of shear from the blocking to the finish material. Blocking of the same depth as the truss heel height should be required.

Illustrating the requirement of split-level ties in the case of a discontinuous roof, Figure 5.5 shows a one-story house with a flat roof at two elevations. The end rafter of the lower roof is attached to the side of the studs and would be prone to separation in an earthquake. A detail similar to 11/4 should be required because the ceiling is, in all likelihood, to be attached to the bottom of the rafters. A steel strap could be substituted for the 2 x 4 shown below the rafters.

It is easy to understand why the post shown in Figure 5.6 is offset from the beam. The post was inserted downward through the cells of the block. The beam location required that it be set slightly farther back than the module of the block would allow. This type of connection is very likely to fail during an earthquake, thereby allowing the entire second floor framing to drop. It is recommended that such beams be placed in pockets within the wall rather than on wood posts buried in the wall. A positive connection can then be made between the beam and the masonry by dropping a long anchor bolt through the beam into a grouted cell, or by attaching a small clip angle to the side of the beam and to the masonry.

Figures 5.4, 5.5 and 5.6 indicate only representative examples of the types of seismic de-

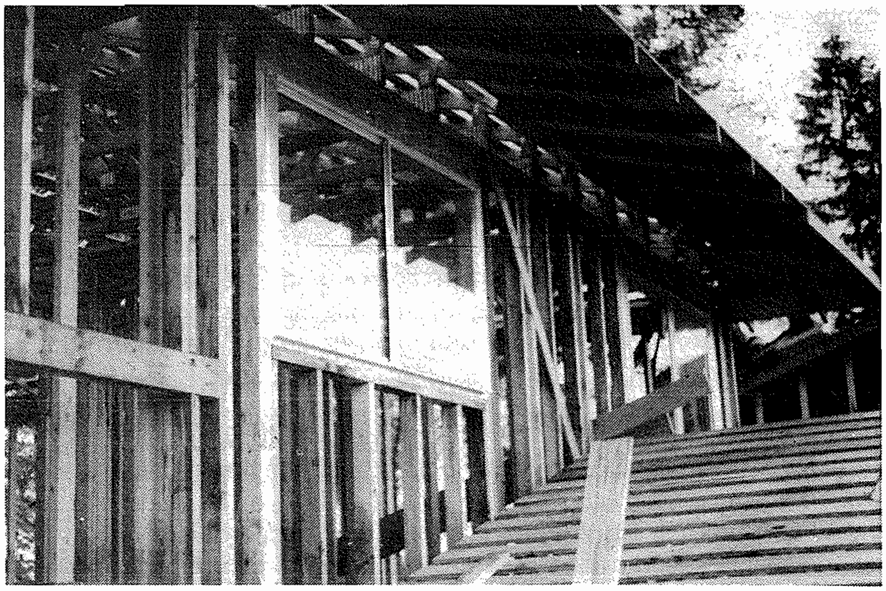


FIGURE 5.4 Blocking at truss ends not full height.

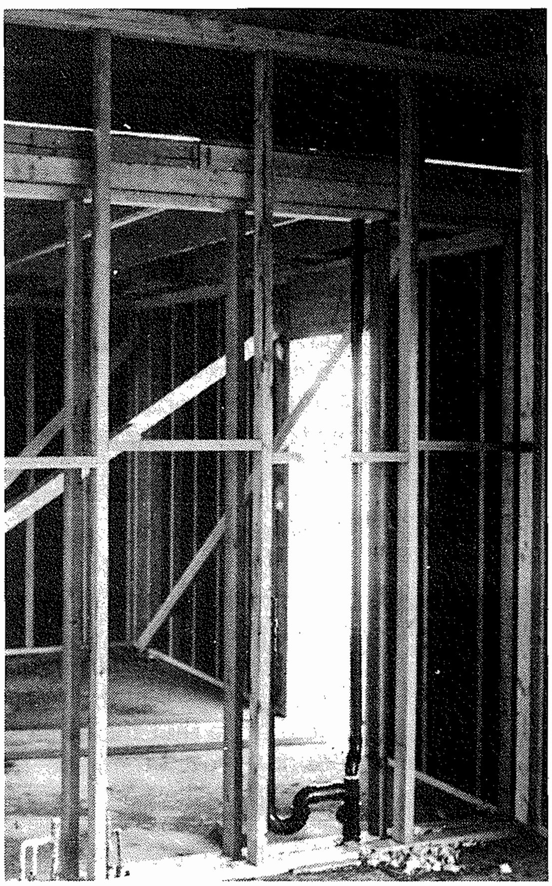


FIGURE 5.5 Discontinuous flat roof requires split-level ties.

FIGURE 5.6 Improper bearing could be even more dangerous during a quake.

sign problems that can be encountered in the field. Although individually engineered detailing could preclude many of these problems, the cost of providing such service for all home designs would be prohibitive. It is incumbent upon the inspector to recognize such situations, despite the fact that they may not have been detailed.

New Construction Requirements

The nailing schedule (Table 4.5), shear transfer to first-floor wood framing (Detail 21/4), chord splice requirements, the use of struts, the fireplace details and the cabinet details are all either new requirements or modifications of existing requirements. In addition, the following deserve special comment for the consideration of the inspector.

Table 4.3: Attachment of Finish Materials. The Design Methodology of the report assumes that all interior walls assist in developing seismic resistance for the residence. In some cases interior walls are designed as shear walls and are then detailed on the plans. All non-designed interior walls must have their finish materials attached in accordance with Table 4.3, although the stud size and spacing need not be as indicated in the heading of that table. Shear walls must be constructed of studs 2 x 4 or larger at 16″ maximum o.c. Occasionally, designed shear walls will require fastenings other than the minimum indicated for the given material. It is required, for example, that gypsum wallboard be fastened with 5d nails at 7″ o.c. in all cases, but the plans may require that the edges of the sheets be blocked and/or that the nailing be 5d at 4″ o.c. The inspector should be aware of the call-outs of special nailing for finish materials and should determine that these requirements are met.

Table 4.4: Installation of Floor Diaphragm. The Methodology considers plywood second-floor diaphragms only. The plywood should be installed in accordance with one of the two patterns indicated in the lower right hand corner of Table 4.4. It is required that all plywood flooring be nailed with 10d nails at 6″ o.c. at the edges of the sheets and at 10″ o.c. to interior supports. No specific detail is supplied for plywood floor installation.

Split-Level Ties: Details 7/4 through 13/4. When the framing is parallel to the wall, split-level ties must be spaced at 16′-0″ maximum o.c. in Zone 2 and 8′-0″ maximum o.c. in Zone 3 as specified in Details 9/4 through 13/4. Details 9/4 and 10/4 indicate a special connection of the tops and bottoms of the studs to which these ties are attached. The inspector should be aware that it is more probable that the top and bottom connections of the studs will be improper or forgotten than that the ties themselves will be omitted. Detail 7/4 is applicable to both Zones and is used when the roof framing is perpendicular to the wall.

Basement Walls: Detail 16/4. This detail not only provides a shear path to the foundation for seismic forces but also provides a reaction at the top of the basement wall for the horizontal pressure of the earth. While it should be up to the plan-checker to determine that the equivalent fluid pressure in the area served does not normally exceed 30 lb/ft^2/ft used in the design of the detail, it will be up to the inspector to determine that the height of earth between the basement floor line and the finish grade does not exceed the 7′-6″ for which this connection is designed.

Stud to Sole Plate Connections: Detail 24/4. The requirement of framing anchors at the base of the studs is entirely new. They are required at exterior corners only in Zone 2 and as shown by the detail in Zone 3. These connections are quite obvious and the inspector should have little trouble in determining if they have been provided.

Hold-Down Anchors: Details 25/4 through 38/4. In addition to the comments made earlier about hold-down anchors, other questions may arise with regard to the installation of these devices. On Detail 25/4, the 8"-long rod extending through the strap would extend out of the footing if the rod were to remain straight. It is acceptable to allow this rod to be bent, provided that the bend does not occur adjacent to the strap itself. If a stud lifting up from the foundation can be visualized, it will be easy to understand why strap-type hold-downs must be installed precisely and tightly.

With hold-downs such as those shown on Detail 34/4, long bolts with a plate adjacent to the head of the bolt are sometimes substituted for the hooks indicated. When used, these plates should be tack-welded to the head of the bolt to prevent them from floating in the concrete during the pour. If the plate is not tight to the bolt head, the device will be ineffective. It is suggested, even encouraged, that the hole in the angle be oversized to as much as 1-1/8" in diameter to allow for tolerance in the placing of the anchor bolts. When oversized holes are used, plate washers should be placed between the angle and the nut. The nut should be tightened securely to ensure that the installation is in tension.

Special Garage Front Wall Details: 45/4 and 46/4. The proper functioning of these details depends on the extension of the garage door header across the top of the wall and on the tautness of straps indicated. When the wall is 4'-0" or wider it should be designed as a shear wall and the special garage wall detail is not to be used. This detail should constructed each side of the garage door whenever the garage door wall is not within 6'-0" of parallel exterior walls of the house, at all split-level houses, and should also be used for all detached garages. Use of a sill plate as a template for the installation of hold-down anchors is encouraged.

Because Detail 15/4 allows the choice of three details for the tying of the slab to the footing, no detail is indicated in the sections shown for the special garage front wall details. One of the connections shown on Detail 15/4 should be chosen and used to provide a tie between the slab and the foundation.

Water Heaters: Detail 49/4. This detail is intended to prevent water heaters from overturning or sliding during earthquakes. It is not necessary that this detail be followed rigorously but a similar tie, capable of acting in any direction, must be provided in all cases. In installations without a wall adjacent to the water heater, ties must be extended from the ceiling or roof framing.

† † † † †

APPENDIX A

EXAMPLE DESIGN

FOR A

SPLIT-ENTRY HOUSE

APPENDIX A EXAMPLE DESIGN FOR A SPLIT-ENTRY HOUSE

Model Home **F** is presented to demonstrate several principles discussed but not shown in Part II, as well as to provide an example of appropriate detailing. The house is a split-entry type home. The plan is wide but not very deep, such that the diaphragm ratio at the roof is almost 2:1. Virtually the heaviest of each type of finish material has been selected so that seismic forces generated are at a maximum allowing a valuable comparison with Model B with its light-weight finishes. Because of the geometry of the structure, the roof is framed from front to rear but second floor joists are "cross-framed" from side to side. Finally, the fireplace is shown as it might be architecturally located with a subsequent structural adjustment (see second floor plan).

The roof is framed with prefabricated trusses and 1/2" plywood sheathing, and roofed with concrete tile. Floor framing is 5/8" plywood sheathing on 2 x 14 joists; interior finish is gypsum lath and plaster; exterior finish is stucco except at the second floor at the front of the residence, which is 3/8" plywood siding.

Separate designs are provided for Zone 3, Zone 2, 15psf wind and 25psf wind. The floor and roof plans without references to the details represent that point at which design would begin. For all designs other than the one for Zone 2, it is quickly determined that it is desirable to provide shear wall in line with the fireplace. In order for this wall to be long enough to be effective, the fireplace is moved from the location shown on any of the second floor plans (and typical foundation plan) to that shown on the first floor plans. Tributary areas are shown on Figures A.3A, B, and C and the pertinent Calc Forms for each design are appended following these figures. Walls B and B′ as shown on Figure A.3C are designed as acting together.

The Hold-down and No hold-down anchors required curves start with walls 6′-0" in height and also extend to 600 lb/ft vertical load. This example provides two conditions for which the curves cannot be used. Wall F is only 4′-0" high and first floor walls A, B, E and F all have loads greater than 600 lb/ft. The formula used to develop the No hold-down anchors required curves is:

$$P = \frac{wL^2}{2h}$$

where: P = allowable horizontal load
L = length of shear wall or "solid" wall
h = height of wall

In Zone 3, for instance, it is necessary to use this formula at lines B and B′ acting together because of the vertical load per foot at line B; and at line F because of both vertical load and the height of the wall.

Although no such condition occurs in this example, the uplift force for which an angle hold-down must be designed can be determined by the formula:

$$U = \frac{P_{act}(h) - \frac{wL^2}{2}}{L - 0.5}$$

where: P_{act} = the design horizontal load
U = the uplift load on the hold-down anchor
L and h = same as above

The formula for strap and framing anchor hold-downs is given in Part III.

Where lightweight materials were used in other models, 15psf wind load frequently governed shear wall design. In this model with its heavy finishes and roofing, Zone 3 loads are greater than those caused by 15psf wind in all cases and greater than the load due to 25psf wind in the longitudinal direction. Because of the methodology, the loads at lines A and D are also greater. The lighter Zone 2 loads are also greater than those caused by 15psf wind for the longitudinal direction.

After the Calc Forms have been completed, applicable details must be referenced. The details shown on the floor plans, roof plan and typical foundation plan (Figures A.1A, B, C and D) are applicable to all designs. Separate foundation plans and, where pertinent, second floor plans are supplied for each design and should be considered as superimposed on the typical corresponding plan.

For Figures A.2A and A.2B 1/2" gypsum board could be used each side of Wall C. While this material is adequate, it is impractical to require that finish material be applied behind the fireplace. It is imperative that the designer visualize the method of construction to avoid problems such as those implied by this condition. In lieu of the gypsum board, plywood, nailed as shown on these figures, has been used. If the room to the rear of the fireplace were finished, this wall could be developed as shear wall in conjunction with the short piece of wall adjacent to the fireplace and the fireplace could be left in its original position. Because the residence is cross-framed, thereby placing greater loads on the shorter shear walls, very few Hold-down anchors are required. Where floor plans make cross-framing practical, this departure from more normal procedure is recommended.

As stated in the text several times, this publication offers a method for dealing with wind load designs only to allow the designer to check wind against seismic loads and no extensive methodology has been developed for dealing with loads other than seismic. This is apparent in the 25psf wind load design where plywood is required at the second floor, Lines A and D, while not required at the first floor in the corresponding locations. This is due to the fact that no interior walls have been used for shear resistance at the second floor while loads to the first floor walls have been based upon their tributary width times the appropriate height of the structure. In such instances, common sense would dictate that, at the minimum, the same plywood and nailing be used at the first floor as well.

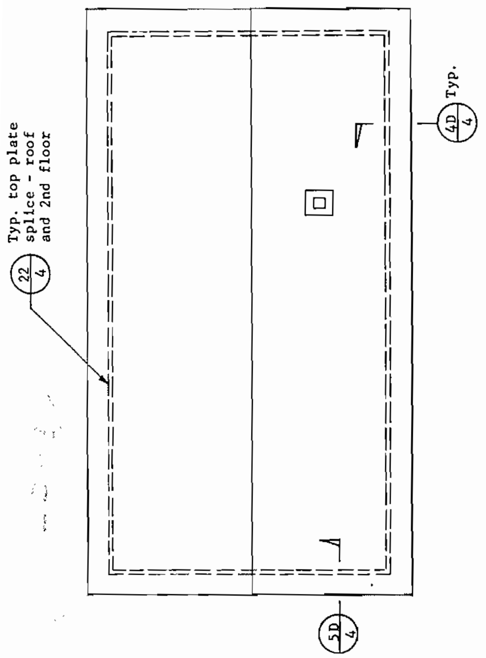


FIGURE A.1A. Roof Plan – Model F.

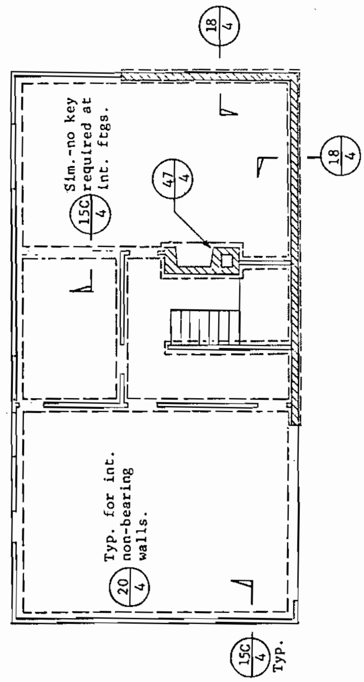


FIGURE A.1B. Typical Foundation Plan – Model F.

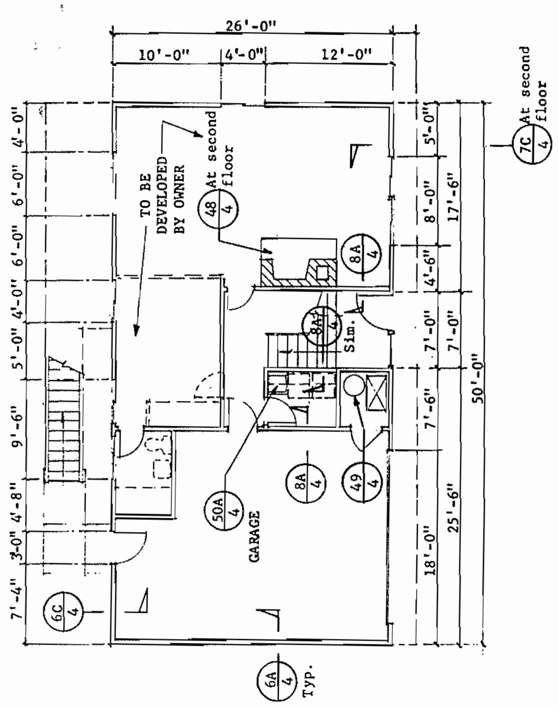


FIGURE A.1C. First Floor Plan – Model F.

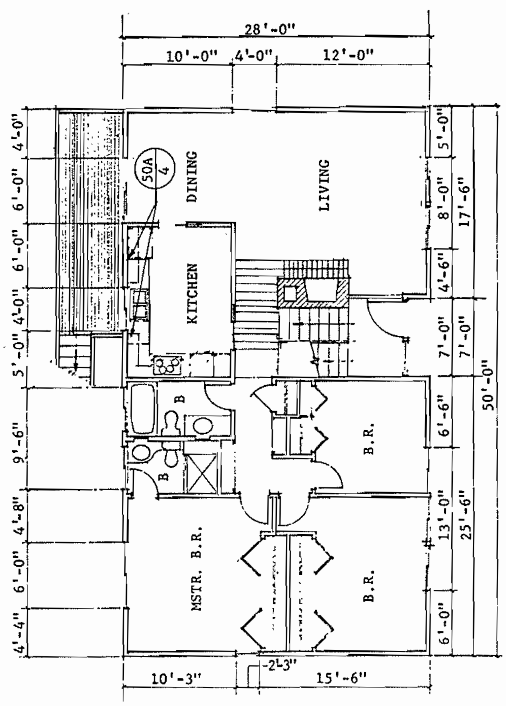


FIGURE A.1D. Second Floor Plan – Model F.

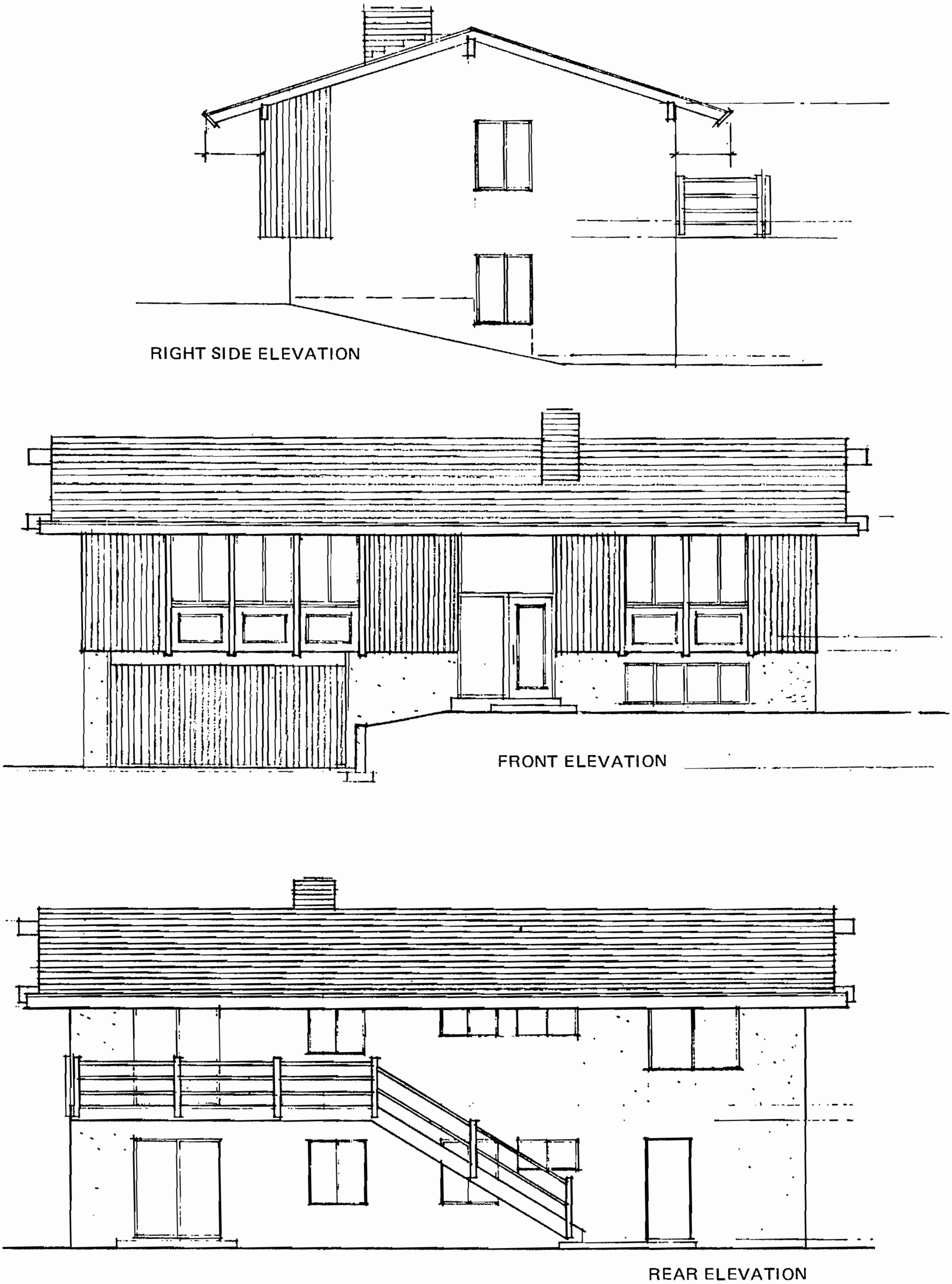


FIGURE A.1E. Elevations – Model F.

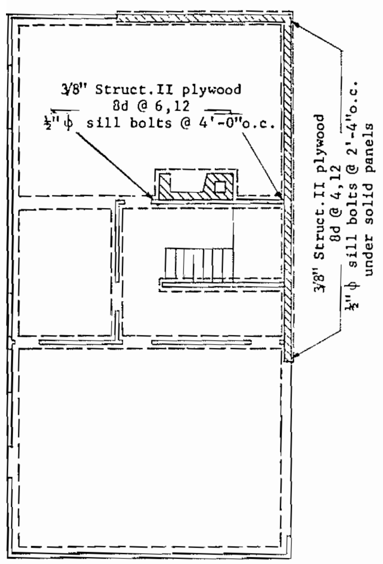


FIGURE A.2A. **Foundation Plan – Model F, Seismic Zone 2.**

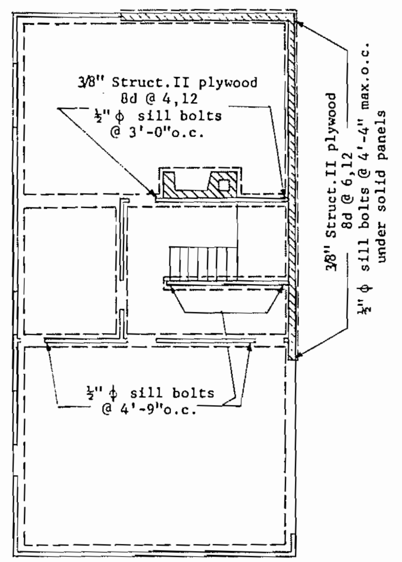


FIGURE A.2B. **Foundation Plan – Model F, 15psf Wind.**

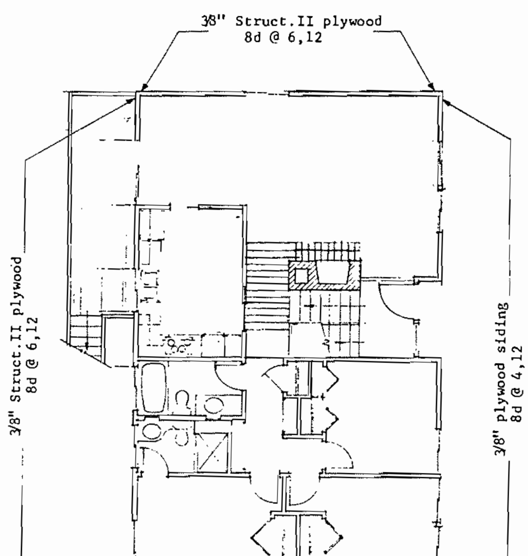


FIGURE A.2C. **Second Floor Plan – Model F, Seismic Zone 3.**

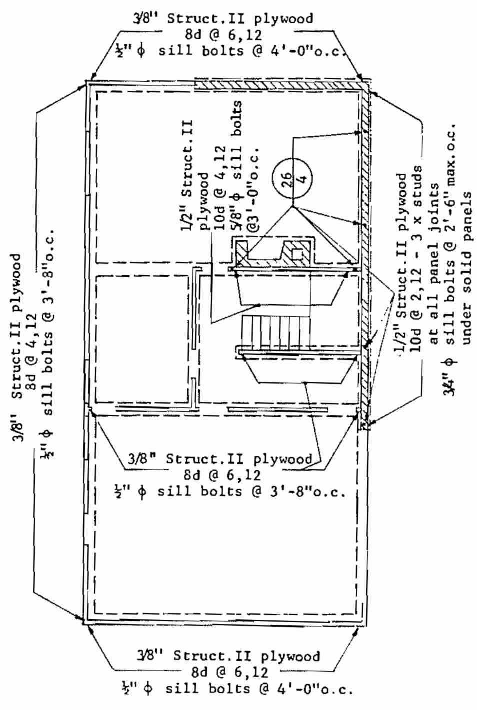


FIGURE A.2D. **Foundation Plan – Model F, Seismic Zone 3.**

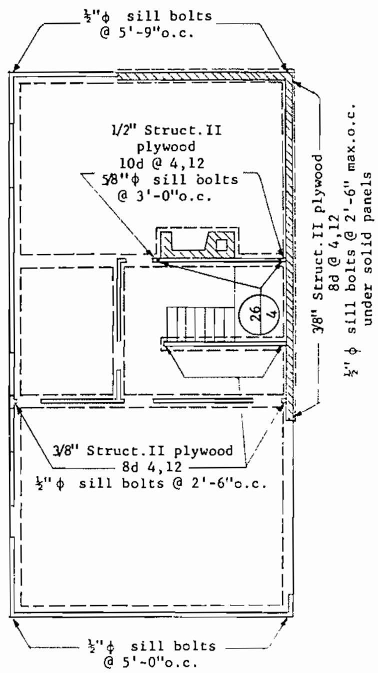


FIGURE A.2E. Foundation Plan – Model F, 25psf Wind.

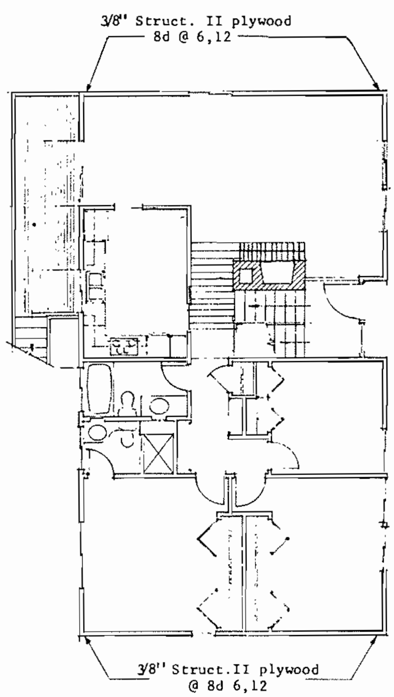


FIGURE A.2F. Second Floor Plan – Model F, 25psf Wind.

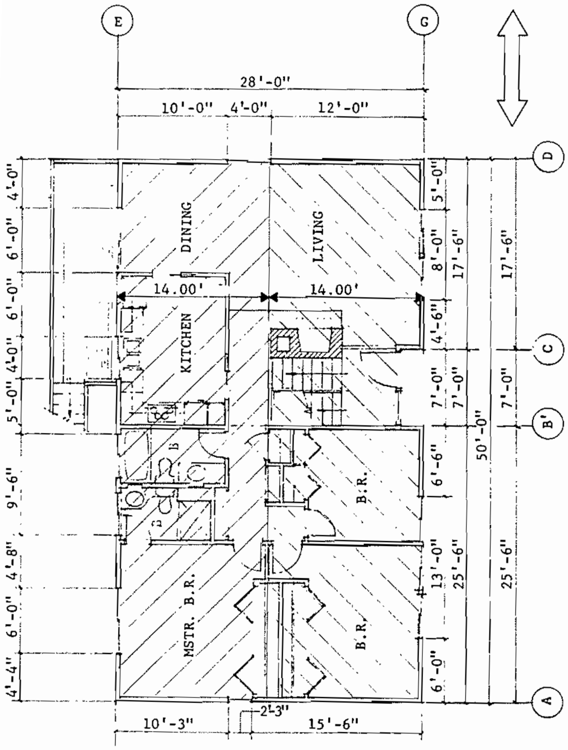


FIGURE A.3A. Second Floor Plan – Model F, Tributary Area.

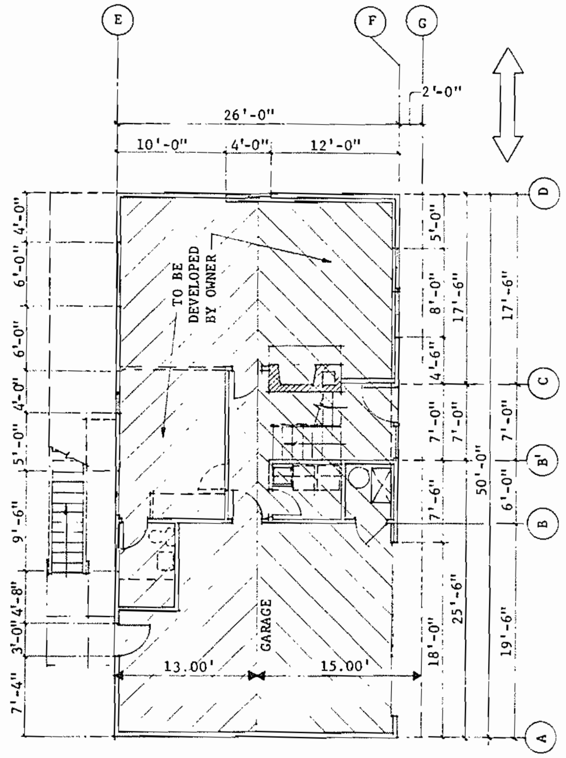


FIGURE A.3B. First Floor Plan – Model F, Tributary Area.

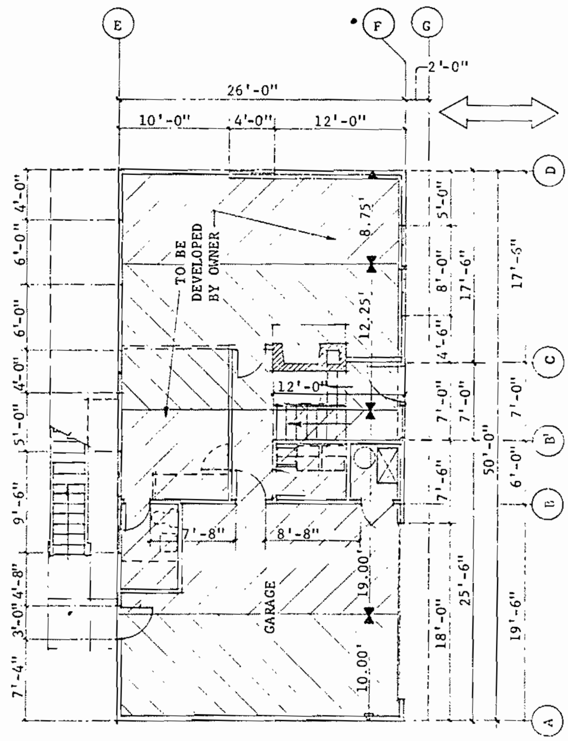


FIGURE A.3C. First Floor Plan – Model F, Tributary Area.

CALC FORM 2

Job EXAMPLE HOMES Model 'F'

ROOF, CEILING AND FLOOR WEIGHTS – SEISMIC UNIT LOADS
(In Pounds Per Square Foot)

ROOF	Material	Actual Weight	Seismic Weight	Material	Actual Weight	Seismic Weight
Roofing	CONC. TILE	16.0	2.133			
Sheathing	1/2" PLYWD.	2.0	0.267			
Framing	TRUSSES	2.0	0.267			
TOTAL		20.0	2.667			

CEILING

Framing	TRUSSES	2.0	0.267			
Finish	LATH & PLSTR	8.0	1.067			
TOTAL		10.0	1.333			

2d FLOOR INT. DECK

Flooring	CARPET	1.0	0.133	DECKING	3.0	0.400
Sheathing	5/8" PLYWD	2.0	0.267	5/8" PLYWD	2.0	0.267
Framing	2x14	5.0	0.667		4.0	0.533
Ceiling	LATH & PLSTR	8.0	1.067			
TOTAL		16.0	2.133		9.0	1.200

CALC FORM 3

Job EXAMPLE HOMES Model 'F'

WALL WEIGHTS – SEISMIC UNIT LOAD SUMMARY
(In Pounds Per Square Foot)

EXTERIOR	TYP. Material	Actual Weight	FRONT Material	Actual Weight	Material	Actual Weight
Framing	WOOD	4.0		4.0		
Exterior Finish	STUCCO	8.0	WOOD SDNG	2.0		
Interior Finish	LATH & PLSTR	8.0		8.0		
TOTAL		20.0		14.0		

INTERIOR

Framing	WOOD	4.0				
Interior Finish	LATH & PLASTER	8.0				
Interior Finish	"	8.0				
TOTAL		20.0				

VENEER LOAD: ____________ lb/ft

TOTAL UNIT LOADS	Roof	2ND FLR CLG.	2ND		DECK	
Ceiling		1.333	2.133		1.200	
Exterior Walls		1.500	3.000			
Interior Walls		0.500	3.000			
TOTAL	2.667	3.333 +	8.133 =	11.467	1.200	

CALC FORM 6

Job EXAMPLE HOMES Model 'F'

TRIBUTARY AREAS

	ROOF AREA		CEILING AREA	
LINE	Length x Width	Area	Length x Width	Area
ROOF				
A	32×27	864	28×25	700
D	32×27	864	28×25	700
E	16×54	864	14×50	700
G	16×54	864	14×50	700
2ND FLOOR				
A			10×28	280
B			9.5×28×1.25+	
			9.5×28	598.5
C			8.75×28×1.25+	
			3.5×28	404
D			8.75×28	245
E			13×50	650
F			15×50	750
PORCH				
C			12.25×6	73.5
D			8.75×6	52.5
E			6×21	126

CALC FORM 7

Job EXAMPLE HOMES Model 'F' – ZONE 3

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
ROOF A	864	2.667		____	700	3.333	2304 2333 4637
D F.P.	864	"		2304 600 2904	700	"	2904 2333 5237
E	864	"		____	700	"	2304 2333 4637
G F.P.	864	"		2304 600 2904	700	"	2904 2333 5237
2ND FLOOR A				____	280	11.467	2304 3211 5515
B				____	598.5	"	6863 6863
C F.P. PORCH				600 600	404 73.5	" 1.200	600 4633 88 5321
D PORCH				____	245 52.5	11.467 1.200	2904 2809 63 5776

CALC FORM 7

Job EXAMPLE HOMES Model 'E' - ZONE 3

SEISMIC LOADS

LINE	Roof Area (sf)	Unit Roof Load (psf)	Other Load (lbs)	Total Roof Load (lbs)	Ceiling Area (sf)	Unit Ceiling Load (psf)	Total Load (lbs)
2ND FLOOR E PORCH					650 126	11.467 1.200	2304 7454 151 9909
E F.P.				2904 600 3504	750	11.467	3504 8600 12,104

MODEL "F" – VERTICAL LOADS/FOOT

ROOF:

LINES A & D

W = ROOF – 3×20 = 60
CLG – 1×10 = 10
WALL – 8×20 = 160
230#/'

LINES E & G

W = ROOF – 16×20 = 320
CLG – 14×10 = 140
WALL – 8×20 = 160
620#/'

2ND FLOOR

LINE A

W = RF LD – = 230
2ND – 10×36 = 360
WALL – 8×20 = 160
750#/'

LINE B

w_1 = 2ND – 12.5×36 = 450 w_2 = 17×36 = 612
WALL – 8×20 = 160   160
610#/'   772#/'

LINE B'

W = 2ND – 3×36 = 108
WALL – 8×20 = 160
268#/'

2ND FLOOR

LINE C

w = 2ND – 8.75x36 = 315
WALL – 8x20 = 160
475#/1

LINE D

w = 2ND – 8.25x36 = 297
WALL – 8x20 = 160
457#/1

LINES E & F

w = RF LD = 620
WALL – 8x20 = 160
780#/1

CALC FORM 8

Job EXAMPLE HOMES   Model 'E' - ZONE 3

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
ROOF						
A	4637	25.75	180.1	——	OK	L=28' OK
D	5237	22.0	238.0	——	3/8" ST II 8d@6,12	L=28' OK
E	4637	24.0	193.2	——	3/8" ST II 8d@6,12	L=40' OK
G	5237	22.0	238.0	——	3/8" SDNG 8d@4,12	L=25.5' OK
2ND FLOOR						
A	5515	26.0	212.1	1/2@4-0	3/8" ST II 8d@6,12	L=26' OK
					200	
B & B'	6863	28.33	242.3	1/2@3-8	3/8" ST II 8d@6,12	OK
	L=12-0	w=268	$\frac{268 \times 12^2}{2 \times 8}$	= 2412		
	8.8	610	$\frac{610 \times 8.6^2}{2 \times 8}$	= 2864		
	7.8	772	$\frac{772 \times 7.6^2}{2 \times 8}$	= 2836		
			3×100	= 300		
				8412		
C	5321	12.0	443.4	5/8@3-0	1/2" ST II 10d@4,12	2 BO FRNG, ANCHS.
	w=475#/1 ;	P_{3U}	= 3625	+ 0.75×	875 = 4	281
		$U = \frac{8}{12}$	(5321 −	4281) =	693	
					180	
D	5776	22.0	262.5	L=26' v=222.1 1/2@4-0	3/8" ST II 8d@6,12	L=26' OK
E	9909	27.0	367.0	L=41-0 v=241.7 1/2@3-8	3/8" ST II 8d@4,12	L=29' OK

CALC FORM 8

Job EXAMPLE HOMES Model 'F'- ZONE 3

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material	Overturning
2^{ND} FLOOR						
F	12,104	17.0	712.0	3/4@2.6	1/2" ST II 10d@2,12	2 BO FRAG. ANCHS.
	L= 7.6	H=4.0	w=780	$\frac{780L^2}{2\times4}$=	5484	
	5.0	"			2438	
	4.6	"			1974	
				3x100=	300	
					10196	
	$U=\frac{4}{17}(12104-10196)=4489$					

CALC FORM 8

Job EXAMPLE HOMES   Model 'E' - ZONE 2

SHEAR WALL DESIGN

LINE	Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
ROOF						
A	2319	25.75	90.1	—	OK	L=28' OK
D	2619	22.0	119.0	—	OK	L=28' OK
E	2319	24.0	96.6	—	OK	L=40' OK
G	2619	22.0	119.0	—	OK	L=25.5' OK
2ND FLOOR						
A	2758	26.0	106.1	½@6	OK	L=26' OK
					200	
B&B'	3432	28.33	121.1	½@6	OK	OK (SEE Z.3)
C	2661	12.0	221.8	½@4.0	3/8" ST.II 8d@6,12	OK
	w=475#/'	P_{ALL}	=3625	+0.75×	875 =	4281#
					180	
D	2888	22.0	131.3	½@6	OK	L=26' OK
E	4955	27.0	183.5	L=41.0 v=120.9 ½@6	OK	L=29' OK
F	6052	17.0	356.0	½@2.4	3/8" ST.II 8d@4,12	OK (SEE Z.3)

CALC FORM 9

Job EXAMPLE HOMES Model 'E' 15PSF WIND

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
ROOF										
A	9.0	25	15	3375		25.75	131.1	—	OK	L=28' OK
D	9.0	25	"	3375		22.0	153.4	—	OK	L=28' OK
E	6.67	14	"	1400		24.0	53.3	—	OK	L=40' OK
									160	
G	6.67	14	"	1400		22.0	63.6	—	OK	L=25.5' OK
2ND FLOOR									180	
A	18.0	10.0	"	2700		26.0	103.8	1/2@6	OK	L=26' OK
									200	
B & B'	18.0	19.0	"	5130		28.33	181.1	1/2@4.9	OK	OK
		SEE SEIS. ZONE 3 - P_{ALL} = 8412#								
C	18.0	12.25	15	3308		12.0	275.7	1/2@30	3/8" ST II 8d@4.12	OK
	w = 475#/l P_{ALL} = 3625 + 0.75 × 875 = 4281									

CALC FORM 9

Job EXAMPLE HOMES **Model** 'E' - 15 PSF WIND

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
2ND FLOOR										
D	18.0	8.75	15	2363		22.0	107.4	½@6	OK	L=26' OK
E	15.67	13.0	"	3055		27.0	113.1	½@6	OK	L=29' OK
F	11.67	15.0	"	2625		17.0	200.3	½@4.4	3/8" ST II 8d@6,12	OK
	4.0	13.0	"	780						
				3405						
		SEE SEIS. ZONE 3 - P_{ALL}=10,196								

CALC FORM 9

Job EXAMPLE HOMES **Model** 'E' - 25 PSF WIND

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
ROOF										
A	9.0	25	25	5625		25.75	218.4	—	3/8" SI II 8d@6,12	L=28' P_{ALL}=9750+
D	9.0	25	"	5625		22.0	255.7	—	3/8" SI II 8d@6,12	L=28' OK
E	6.67	14	"	2335		24.0	97.3	—	OK	L=40' OK
									160	
G	6.67	14	"	2335		22.0	106.1	—	OK	L=25.5' OK
2ND FLOOR									180	
A	18.0	10.0	"	4500		26.0	173.1	1/2@50	OK	
									200	
B & B'	18.0	19.0	"	8550		28.33	301.8	1/2@2-6	3/8" SI II 8d@4,12	OK
		SEE SEIS. ZONE 3 –			P_{ALL} =	8412 +	1.6% OVER			
C	18.0	12.25	25	5513		12.0	459.4	5/8@30	1/2 SI II 10d@4,12	2 BO FRIG. ANCHS
	w=475#/'	P_{ALL}=3625		+0.75×	875 =	4281				
		$U=\frac{8}{12}$(5513		-4281)	= 821.3					

CALC FORM 9

Job EXAMPLE HOMES Model 'F' - 25 PSF WIND

WIND LOAD AND SHEAR WALL DESIGN

LINE	Tributary Height (ft)	Tributary Width (ft)	Wind Load (psf)	Total Wind Load (lbs)	Total Seismic Load (lbs)	Wall Length (ft)	Shear Per Foot (lb/ft)	Sill Bolts	Shear Material 180	Overturning
2ND FLOOR										
D	18.0	8.75	25	3938		22.0	179.0	L=26' v=151.5	OK	L=26' OK
								1/2@5-9		
E	15.67	13.0	"	5093		27.0	188.6	L=41' v=124.2	OK 4.8% OVER	L=29' OK
								1/2@6		
F	11.67	15.0	"	4376		17.0	333.9	1/2@2-6	3/8" ST I Ed@4,12	OK-SEE SEIS. Z3
	4.0	13.0	"	1300						
				5676						

APPENDIX B

ADDITIONAL EXAMPLES

FOR DETERMINATION

OF TRIBUTARY AREAS

APPENDIX B ADDITIONAL EXAMPLES OF TRIBUTARY AREAS

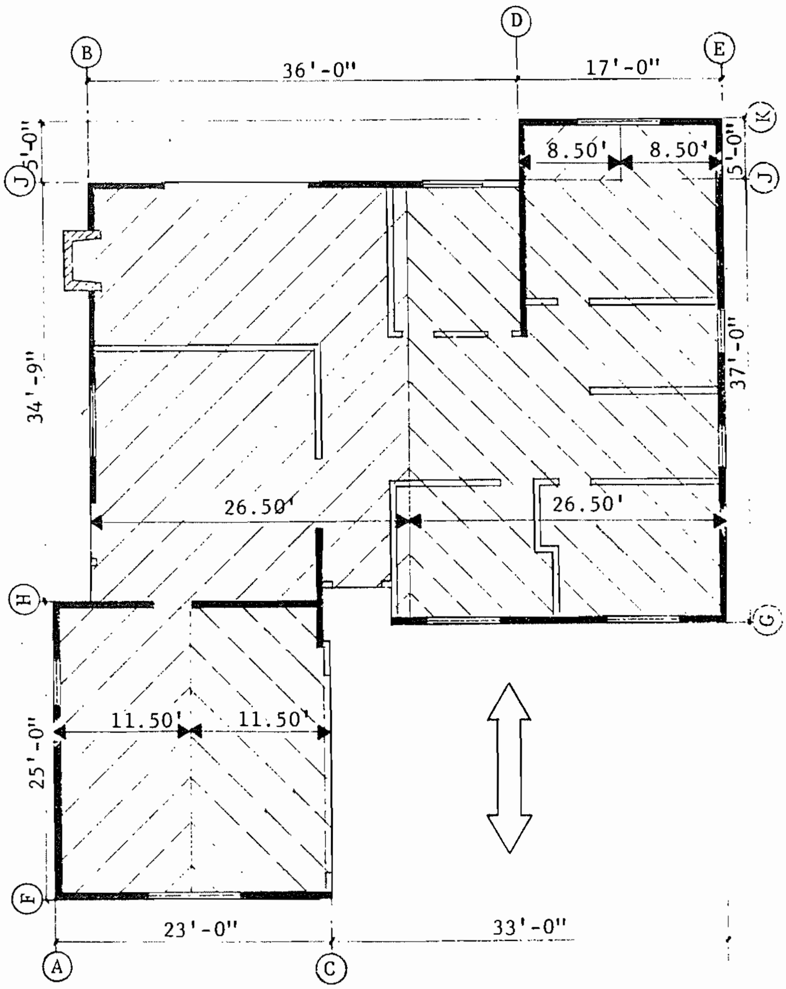
Appendix B presents additional examples of determination of tributary area. These include seven one-story houses (Figures B.1 through B.7), three one- and two-story combination houses (Figures B.8 through B.10), two split-level houses (Figures B.11 and B.12) and one house with a design beyond the scope of this publication (Figure B.13). The examples should lead to a fuller understanding of the techniques involved in assaying shear wall locations and determining tributary lengths and widths; they may also be helpful in terms of comparison with plans being developed which have similar conditions. The reader may also wish to examine the houses for alternative interpretations on the location of shear walls and the figuring of tributary areas. In most cases the Design Methodology does not allow much latitude in this area but correct interpretation of the requirements is essential in order to achieve proper and economic design.

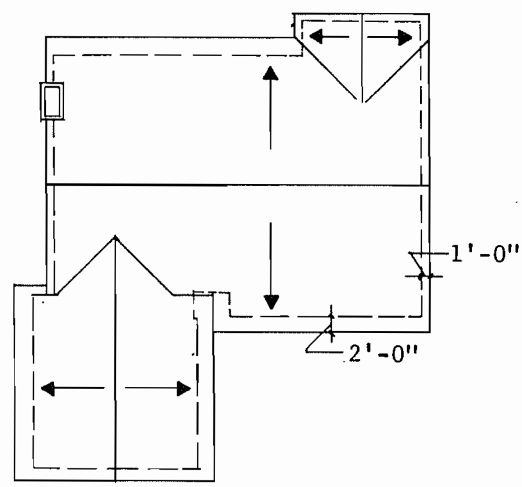
Floor plans discussed are essentially identical with those of homes that incurred slight to severe damage in the San Fernando Earthquake. Since many of the houses were not engineered, adequate shear wall is not always present at each line of resistance. As a most important consideration, however, these fairly typical houses in nearly every instance provide walls that may be used as shear walls in accordance with the Design Methodology. This is true despite the fact that the homes were located in an area where large amounts of glass are utilized. It will be noted that every home shown contains an attached garage. In virtually every case, without the attached garage the location of shear walls and determination of tributary areas would be much simpler. In the few cases where this is not true, the garages are indented into the home and the floor plan would probably not be drawn in the manner shown if the garage were not attached.

All tributary widths are determined using the exact method assuming a wall thickness of 6". In other words, all wall thicknesses (or half thicknesses) have been deducted from diaphragm span, the clear span divided in half, and then appropriate wall thickness added to each width. As stated in Part II, this method is more accurate but need not be used in actual practice. The importance of the examples lies primarily in the identification of lines of resistance and the proper application of the rules for determining tributary areas rather than in the exact determination of the width of the areas to the last inch. Indeed, the exact and approximate methods will never vary by more than three inches for any single width.

Although comments accompany each figure, references to applicable Design Sections are usually not made and, for the most part, calculations are not shown. The effect of masonry fireplaces on individual lines of resistance is included in the commentary.

FIGURE B.1





Roof Plan

Figure B.1A

1. Lines A and B should be considered as acting together since the distance between them is less than 6'-0".

2. The wall on Line C need be considered only as resisting garage load. The wall extends inside the house and the portion inside the house may or may not be used at the discretion of the designer.

3. The comments above also apply to the portion of wall inside the house on Line D. Other interior walls are not consid-

ered since no portion of these walls comes in contact with the roof.

4. The tributary area to Line C stops at Line H, but the tributary area to Lines A and B together includes one-half of the main body of the house.

5. The wing between D and E is considered as a wing with a 5'-0" x 8'-6" tributary area to the wall on Line D.

6. Fireplace load should always be applied to the wall in which the fireplace occurs – in this case, Line B.

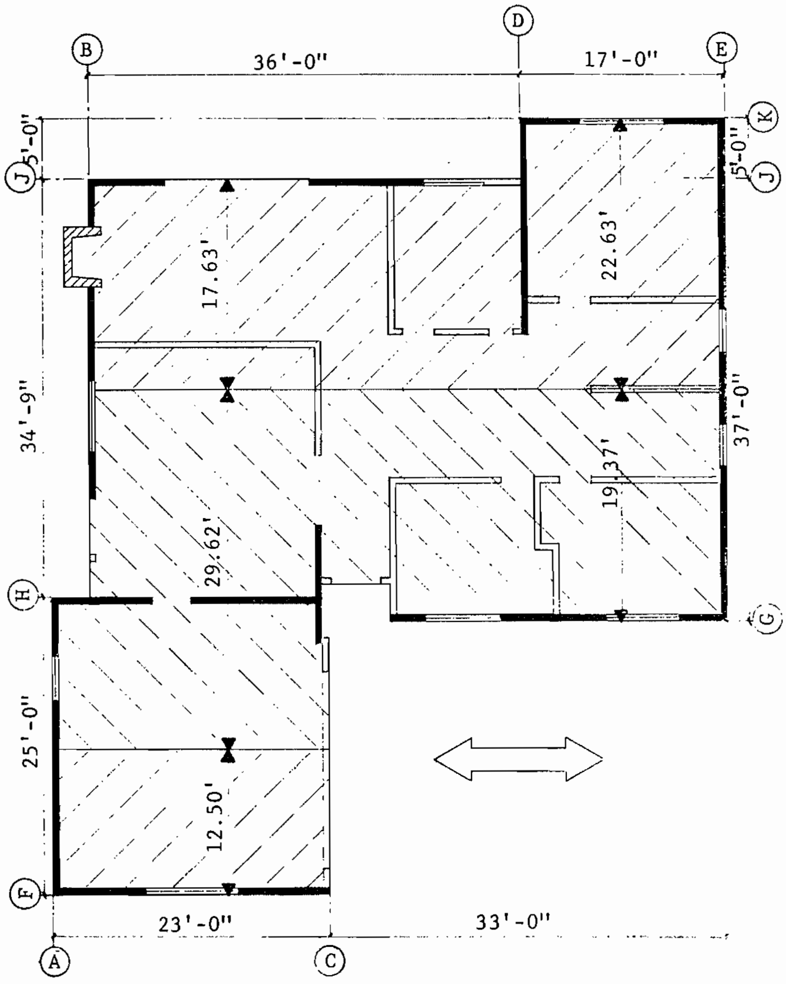


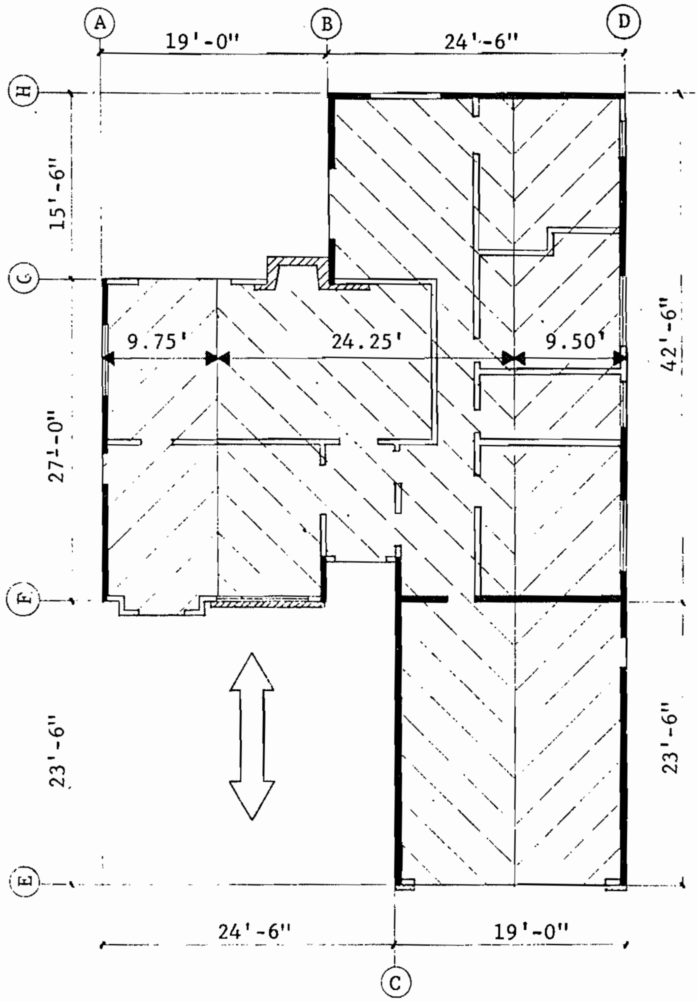
Figure B.1B

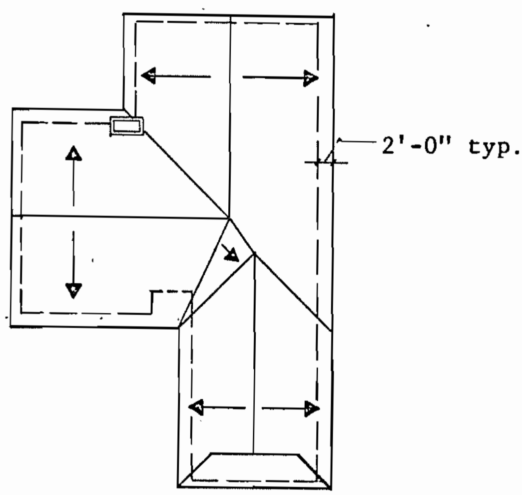
1. The walls on Lines G and H should be considered as acting together as should the walls on Lines J and K. When lines are six feet or less apart, they are always considered as acting together regardless of changes in diaphragm length.

2. Tributary widths are calculated between closest of the lines acting together (3.3A2) – in this case Lines H and J.

3. The fireplace load will be taken by Lines J and K acting together.

FIGURE B.2





Roof Plan

Figure B.2A

1. With the garage excluded, the house is "L" shaped. The portion from A to B or the portion from G to H could be considered a wing. Because of the near symmetry and the roof layout, neither has been so considered.

2. Lines B and C should be considered as acting together.

3. The fireplace load will be taken by lines B and C acting together.

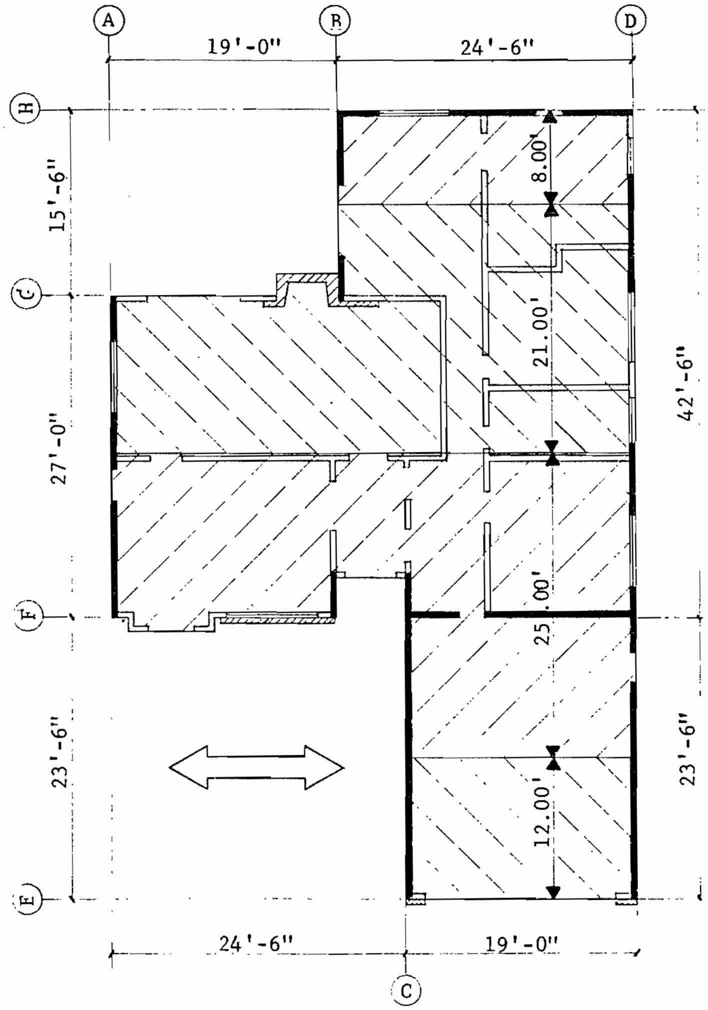


Figure B.2B

1. The garage, if considered as a cantilever, would have a cantilever distance of 23'-6". This is more than half the distance of the back span of 42'-6". Therefore, this garage does not qualify as a cantilever.
2. The special garage front wall detail (Detail 45/4) needs to be utilized at Line E and, for this one-story condition, is considered equivalent to a shear wall.
3. There is no shear wall on Line F at the front of the house, but since the wall between the garage and the main portion of the house is in line at this point, and since the roof for this plan is continuous across the entry, the garage wall may be considered as a portion of the exterior wall and may be used. It is assumed the garage wall follows the contour of the roof, and that load can therefore be transferred directly from the roof to the wall.
4. There is no shear wall provided at Line G. This condition must be corrected. Note that the interior wall on Line G to the right of Line B could be used if the top plates are continuous past the chimney to this portion of wall. The top plates would pick up the load between A and B and act as a strut to carry the load to this wall. The same reasoning as is applied to item 3 can be used to consider this interior wall as a portion of the exterior wall. The interior portion of the wall would not be required to extend to the roof since

the top plates acting as a strut would carry the load to it. If the chimney interrupts the top plates, the opening would have to be moved or decreased in width in order to provide at least one 4'-0" minimum long wall. The short length of wall available is further complicated by the load of the fireplace which must be resisted by this line.

FIGURE B.3

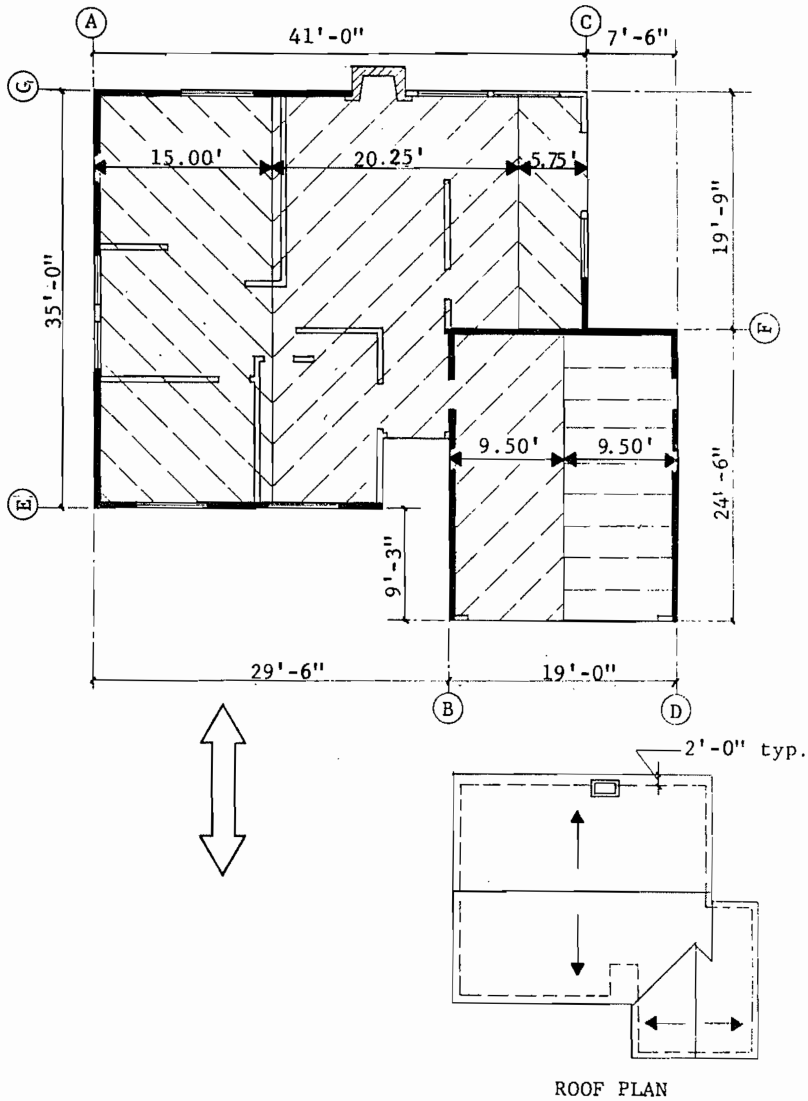


Figure B.3A

1. The wall on Line B should be considered a shear wall for the house as well as the garage. It is assumed that the wall follows the roof level upward (refer to roof plan) in order to act as a fire wall.

2. The wall on Line C is considered for the small portion of load from the house only. The Load from the garage should be considered as being taken to Line D. Per 3.2B these lines

cannot be considered as acting together. (24.5′ is less than 2/3 of 44.25′)

3. The fireplace load will be taken by Line B.

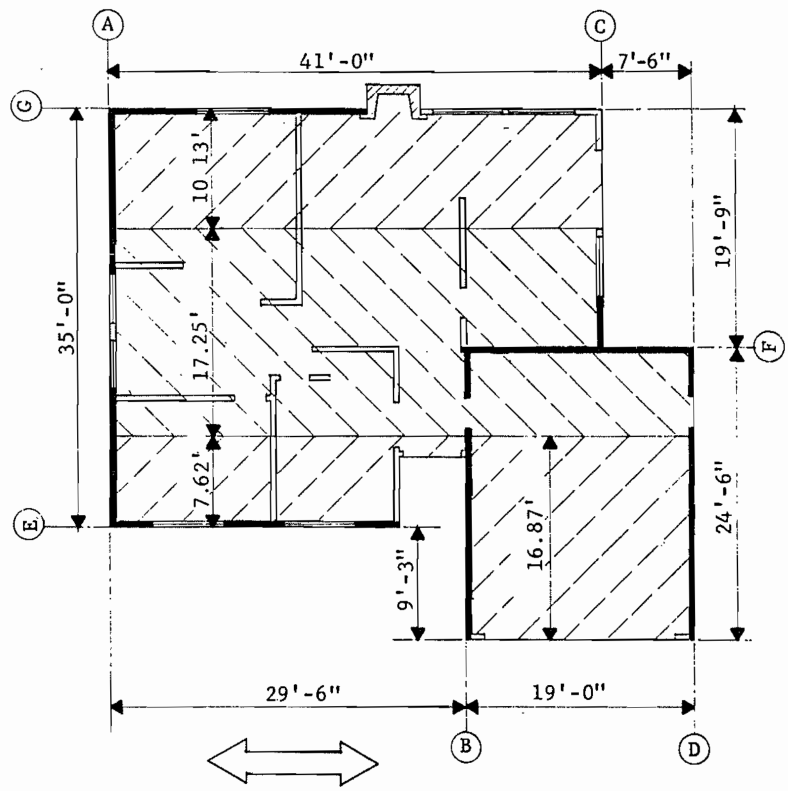


Figure B.3B

1. The walls on Line E can be developed to provide the resistance for the garage. The garage can therefore be considered a 9′-3″ cantilever and need not have the special garage detail at its front. If the offset between Lines C and D were more than half the width of the garage, Line G would not be considered in figuring backspan length but Line F would prove to be too close and Detail 45/4 would then be necessary.

2. The cantilever distance of 9′-3″ is less than twice the distance from Line E to Line G of 35′-0″. Design Requirement 2.1B3 is therefore met.

3. When using the cantilever considerations above, cantilever distance may be calculated from the floor plan if roof overhangs are nominal (1—3 feet). When roof overhangs are larger, the actual dimensions of the roof should be considered.

4. The wall on Line F is at least as long as any other wall in the house for this direction of load, and should be used for the main body of the house as well as the garage.

5. Fireplace load will be taken by Line G.

FIGURE B.4

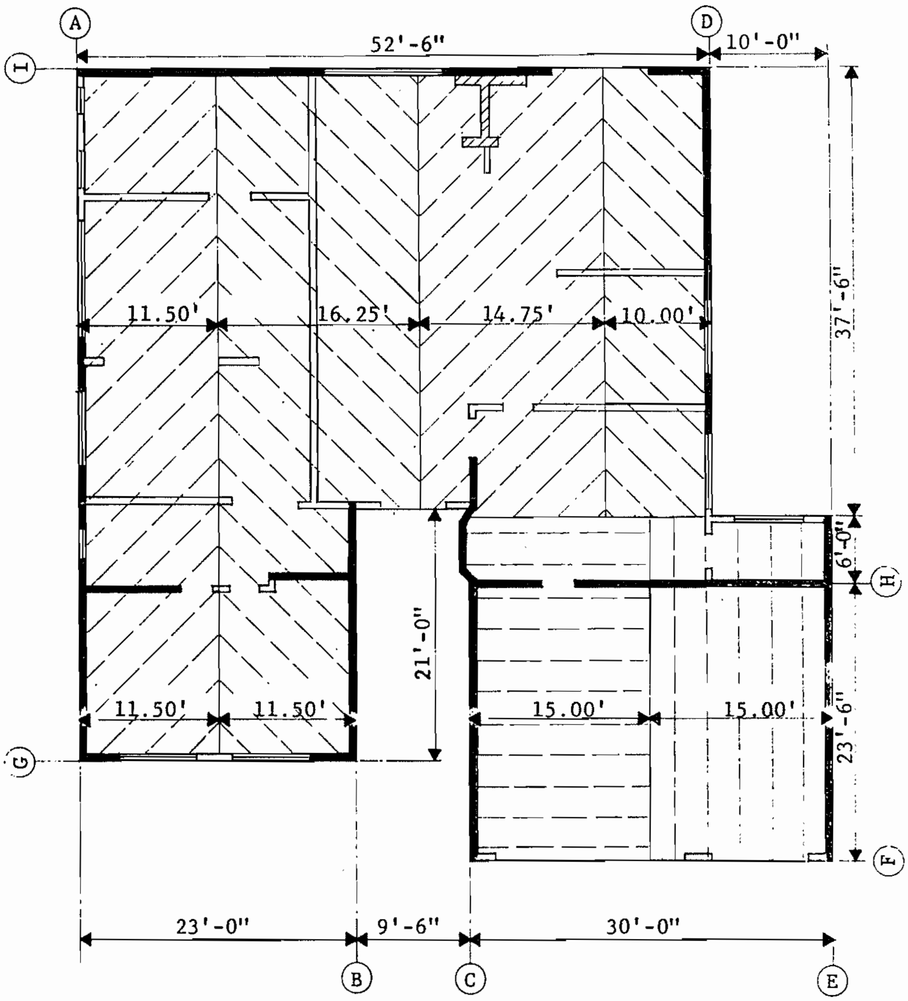
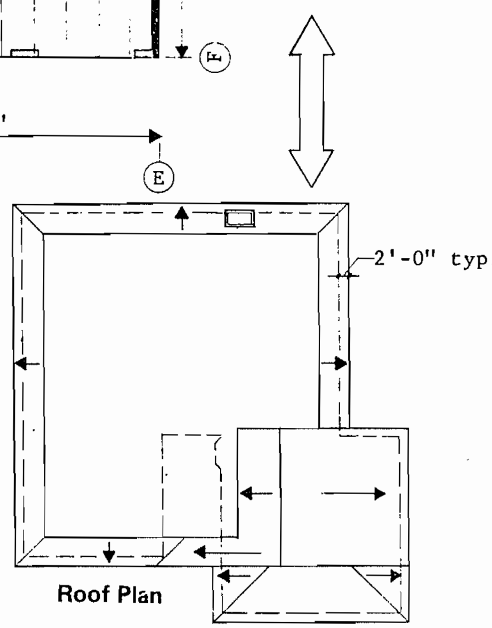


Figure B.4A



1. The walls on Lines B and C are both over fifteen feet in length and should be considered as resisting load from the main portion of the house as well as from each wing. Since the roof extends across at Line G, the tributary width division line between B and C will actually be extended downward on the roof to below Line G.

2. A strut connection at Lines B and C need not be developed since they intercept such a large portion of the main body of the roof.

3. Since Line D is 10 feet away from Line E, house tributary should be figured to Line D

and garage tributary to Line E. Because of the break in the diaphragm behind the garage as indicated in the roof plan, the tributary length to Line E should extend to the break.

4. Fireplace load will be taken by Line C since it falls wholly within the area tributary to that line.

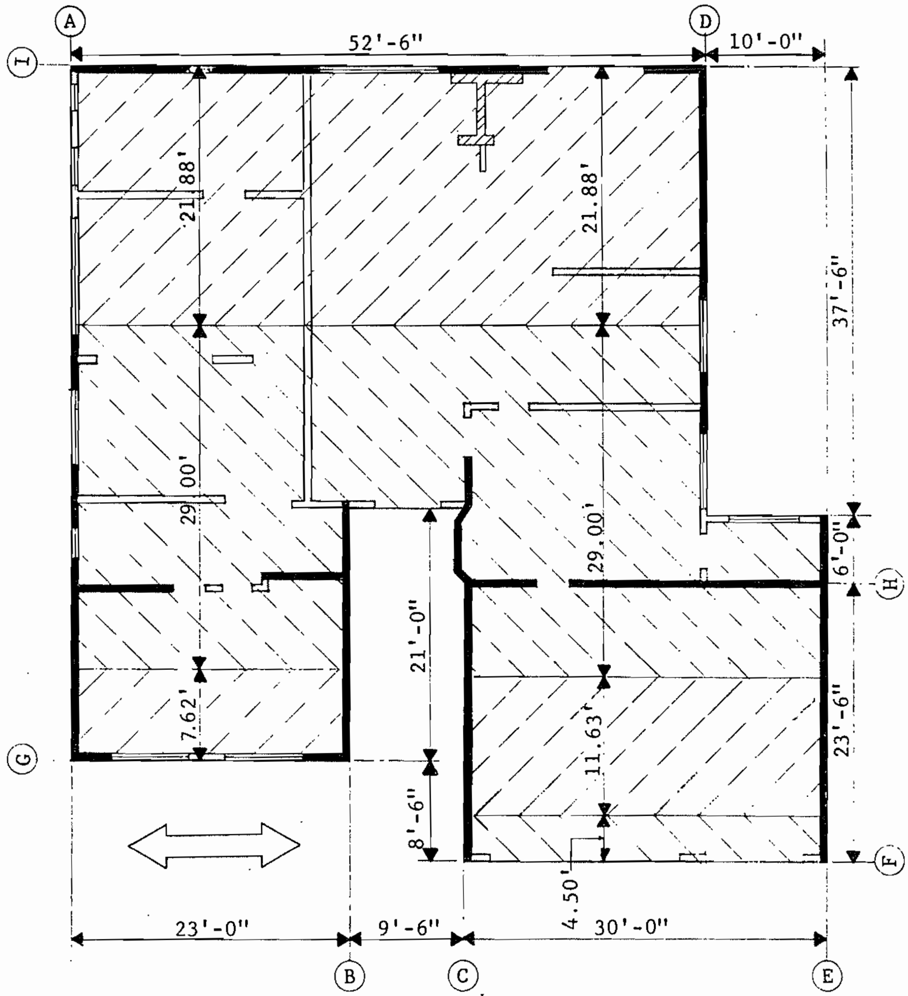


Figure B.4B

1. Since the front garage roof is a discontinuous diaphragm, the roof cannot be cantilevered. The load from the garage should be divided between Lines F and G. The short vertical wall occurring over the garage at Line G will act to transfer the load to the walls on Line G.
2. A small portion of the shear wall on Line H is slightly back of the remainder. Since such a large preponderance of this wall is on Line H, and the offset is so small, all distances have been figured from Line H.
3. No shear wall is present at the front of the garage and the special garage detail must be used.
4. Because of the break in the diaphragm 6 feet to the right of Line H, and because there is no wall used as shear wall at the break, the flat roof diaphragm should be extended to Line H under the gable roof to transfer shear into wall H. The other alternatives would be to use the wall adjacent to the front entrance as shear wall or to provide shear wall at the break.
5. Fireplace load will be taken by Line I.

FIGURE B.5

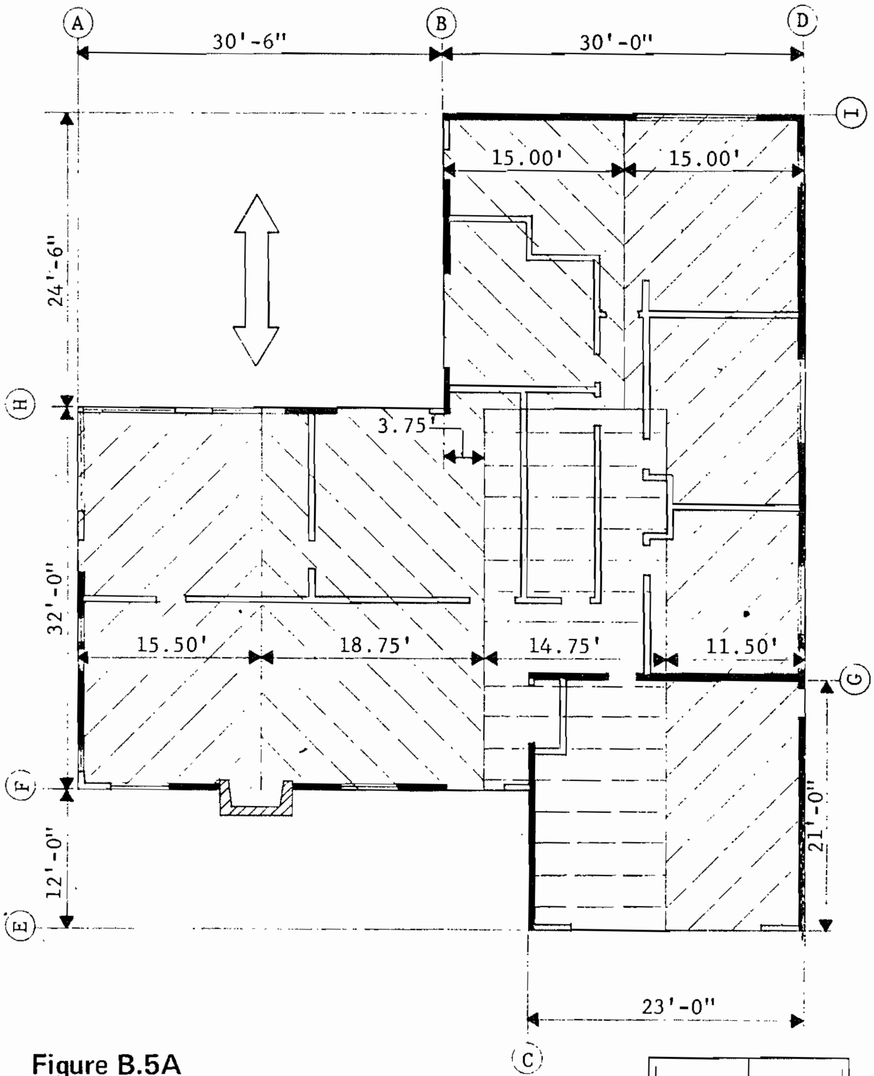
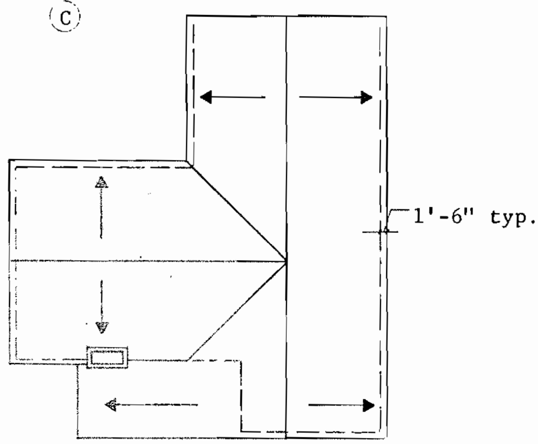


Figure B.5A



Roof Plan

1. The walls on Lines B and C could be designed as acting together such as was done on Figure B.2A (the shorter length of the diaphragm is more than 2/3 the longer length). If considered as shown, however, the general principle stated at the beginning of Design Section 3.3 must be taken into consideration. The wall on Line C is longer than the wall on Line B, but the tributary area to Line B from A to B plus the tributary area of the wing contributes more load to Line B. The tributary area to Line C should therefore

be made as long as possible. Roof directly opposite a wall should always be considered as contributing load to that wall, and therefore the length of the tributary area on Line C is restricted to the length to Line H.

2. If walls B and C were considered as acting together it would not be necessary to jog the tributary area division line at Line H. The line between C and D would extend all the way to Line I. Considering these lines as acting together would be preferable to the solution shown.

3. Fireplace load will be divided equally between Lines A and B.

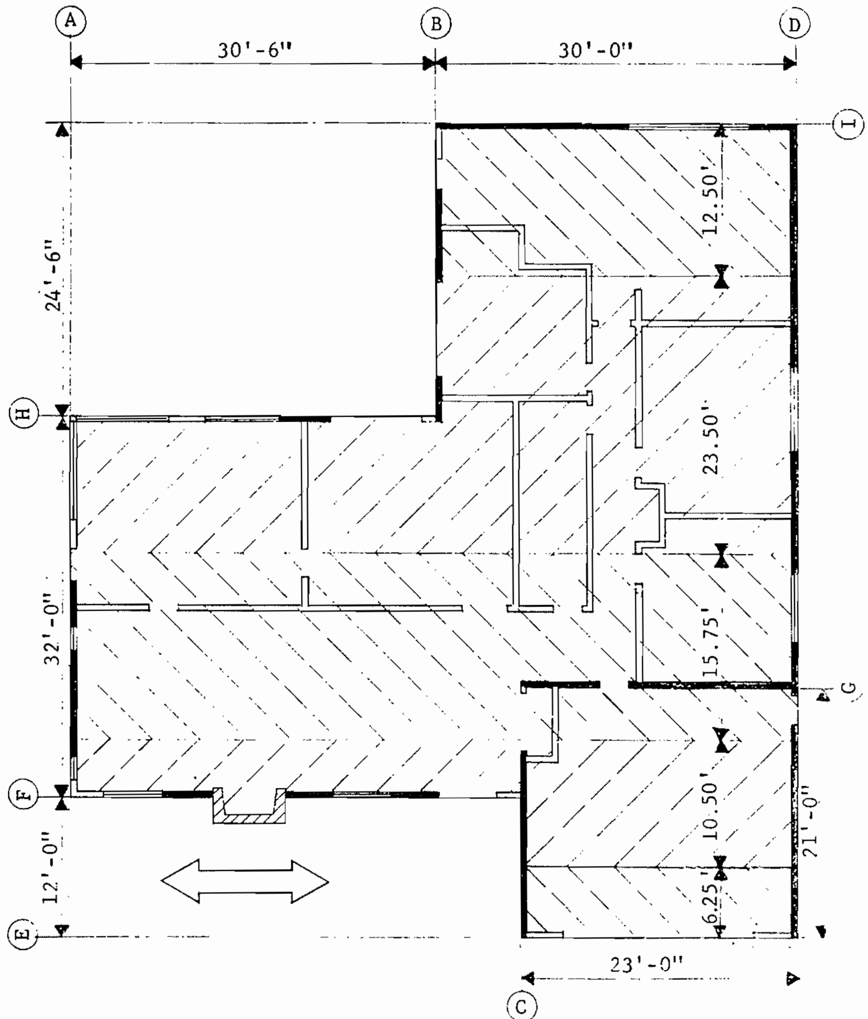


Figure B.5B

1. The diaphragm between Lines E and F could be considered a cantilever. It would probably be more desirable to offset the garage doors such that 4 feet of wall could be provided adjacent to Line C. Tributary widths have been calculated on this basis.

2. There is almost assuredly insufficient shear wall at Line H. Since the interior wall is not in line with the exterior wall, the plates along Line H cannot be utilized as struts to the interior wall and it therefore cannot be used in the manner suggested under Figure B.2B. If it were necessary to maintain all of the openings at Line H, a roof rafter could be used as a strut and connected to the offset interior wall at Line H.

3. Fireplace load will be taken by Line F.

FIGURE B.6

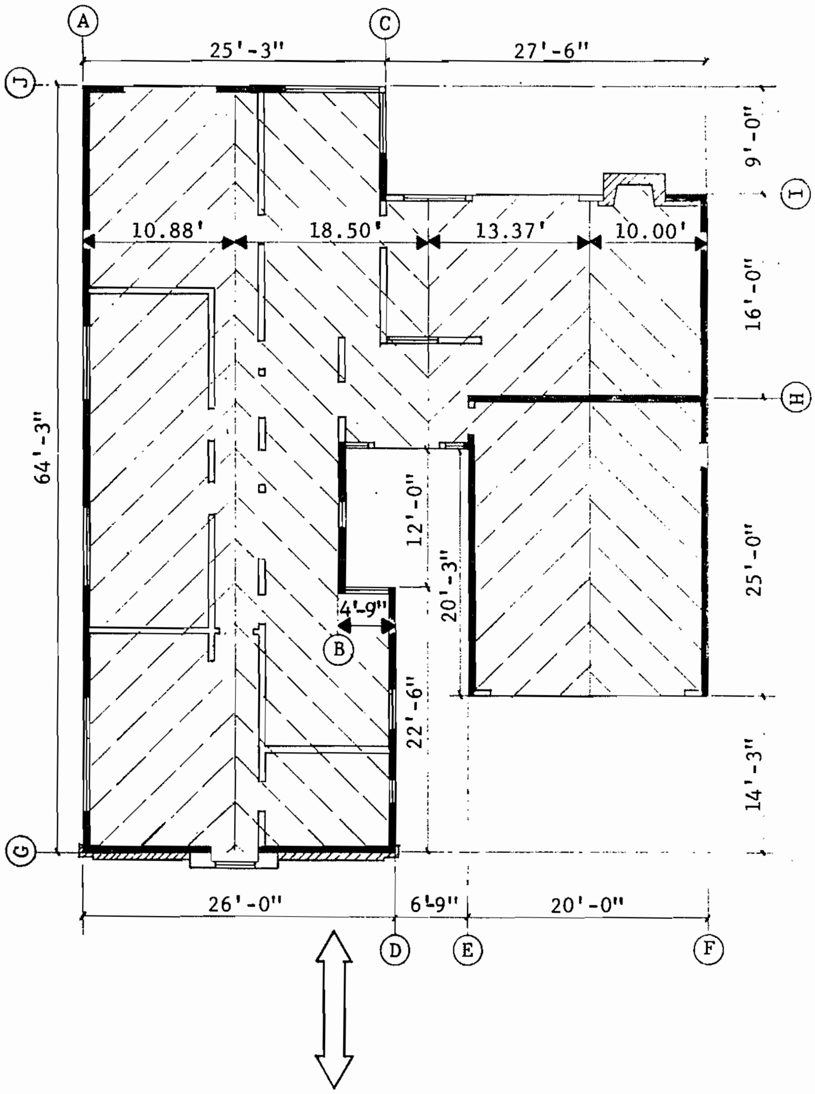
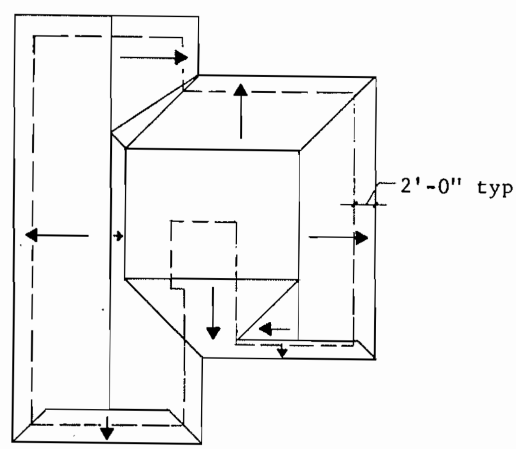


Figure B.6A

1. The walls on Lines B, C and D should be considered as acting together (3.2C) Design Section.

2. The fireplace load will be taken by Line F.



Roof Plan

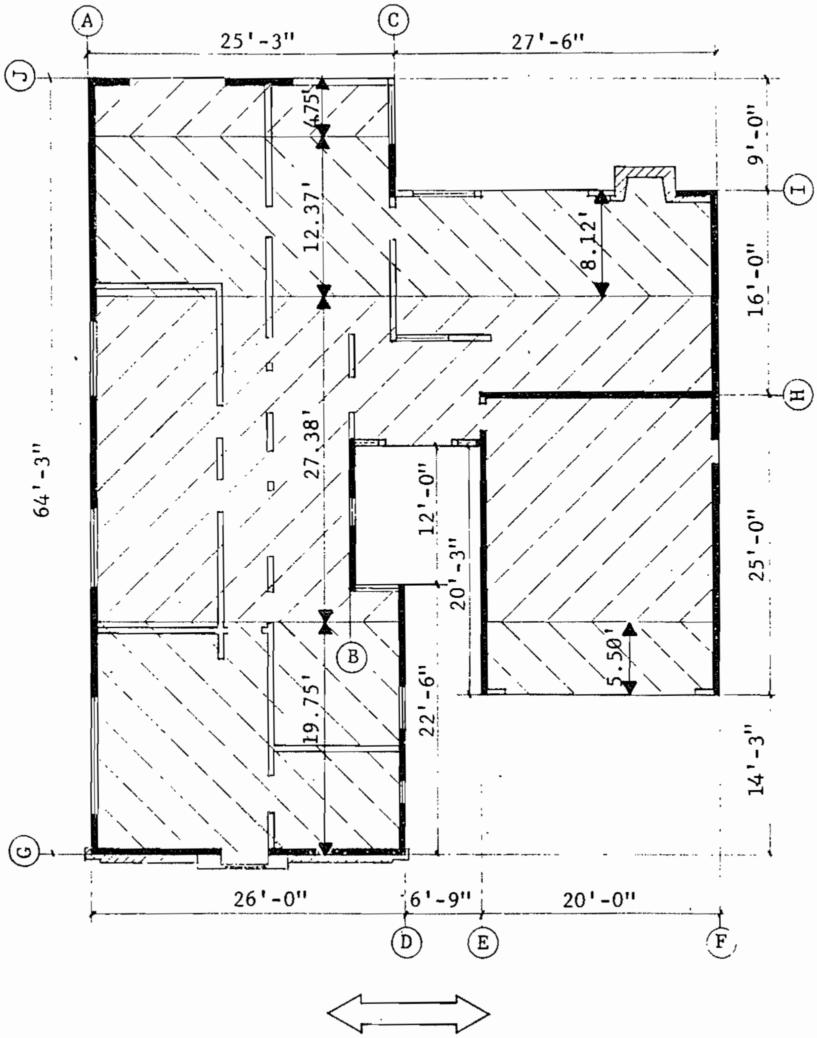


Figure B.6B

1. Since the roof diaphragm is continuous over the entryway, the front of the garage does not require the special garage detail. The load from the front part of the garage should be considered to be taken by Line G.

2. There does not appear to be sufficient shear wall at Line I.

3. Lines I and J may not be considered as acting together. Since the diaphragm breaks and since there is more wall on Line J than there is on Line I, considerable load would have to be transferred through the narrower portion of the diaphragm to Line J. It is partly for this reason that the exception shown in Design Section 3.2B is stated.

4. Fireplace load will be taken by Line I.

FIGURE B.7

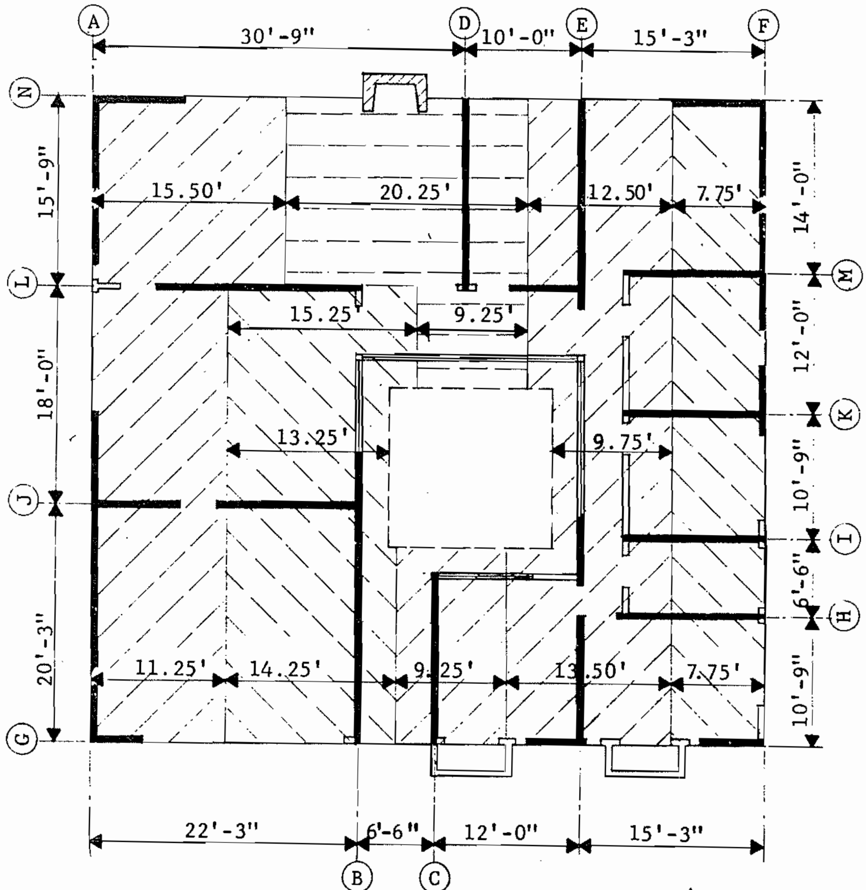


FIGURE B.7A

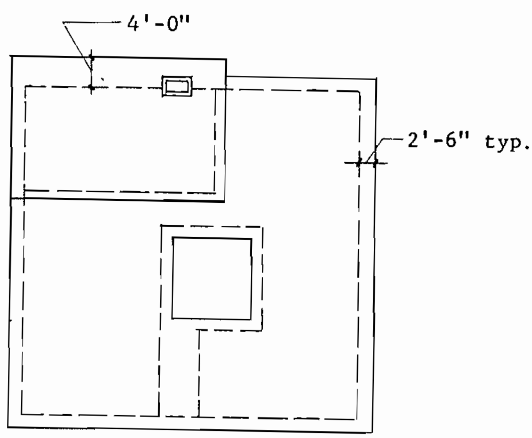
Figures B.7A and B.7B

1. This house has a flat roof and all interior walls extend to the roof. It is therefore necessary that virtually all walls be considered as shear walls as shown.

2. The high roof from A to D and L to N requires that shear walls be located at all four sides of the discontinuous roof (2.3B2)

3. The fireplace load will be taken by the walls on Lines D and N.

4. The only walls which may be eliminated from consideration by virtue of Section



Roof Plan

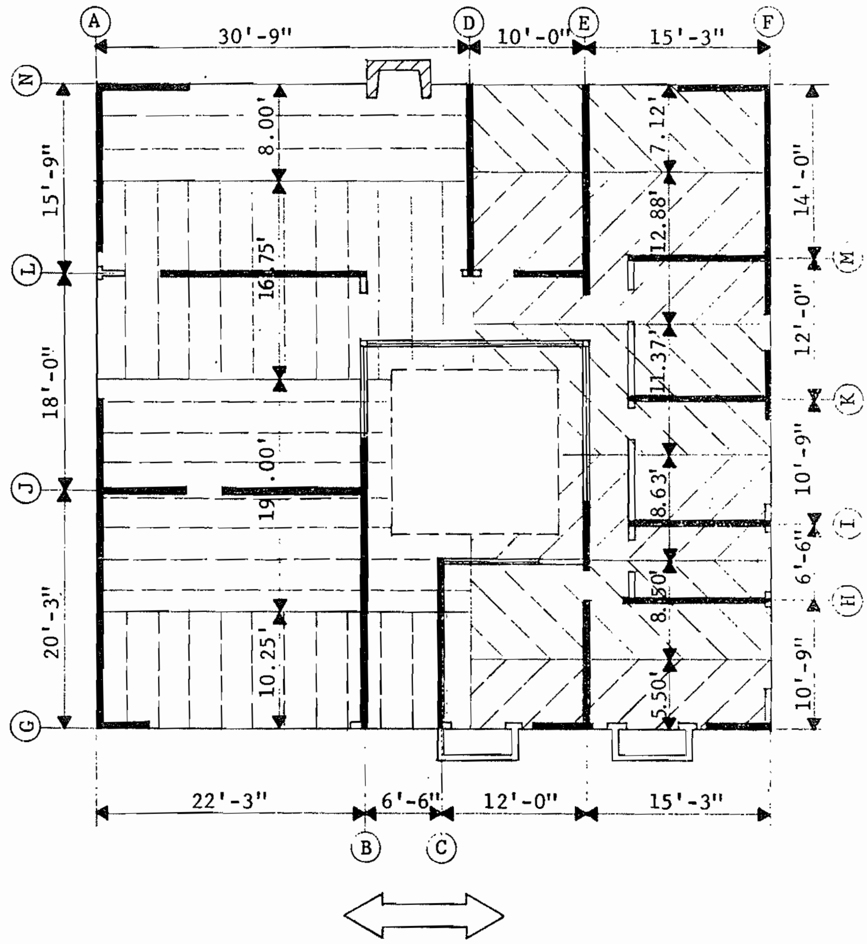


FIGURE B.7B

2.2B2b are the walls between Lines E and F. (Wall C can also be eliminated it it is not considered an exterior wall.)

5. Note that walls perpendicular to the open atrium area have tributary lengths which stop at the edge of the atrium roof and are not considered as acting together with walls at the opposite side of the atrium.

6. Lines L and M should be considered as acting together.

7. The consideration of the general principle stated at the beginning of Section 3.3 must be utilized for Figure B.7C.

FIGURE B.8

Note that the roof plan for this house indicates that the high roof over the second story continues downward over the one story portion in the rear and that therefore the shear walls on Lines C and E are each really two separate walls, since the portion under the second floor extends only to the second floor, while the portion between Lines G and I extends to the roof.

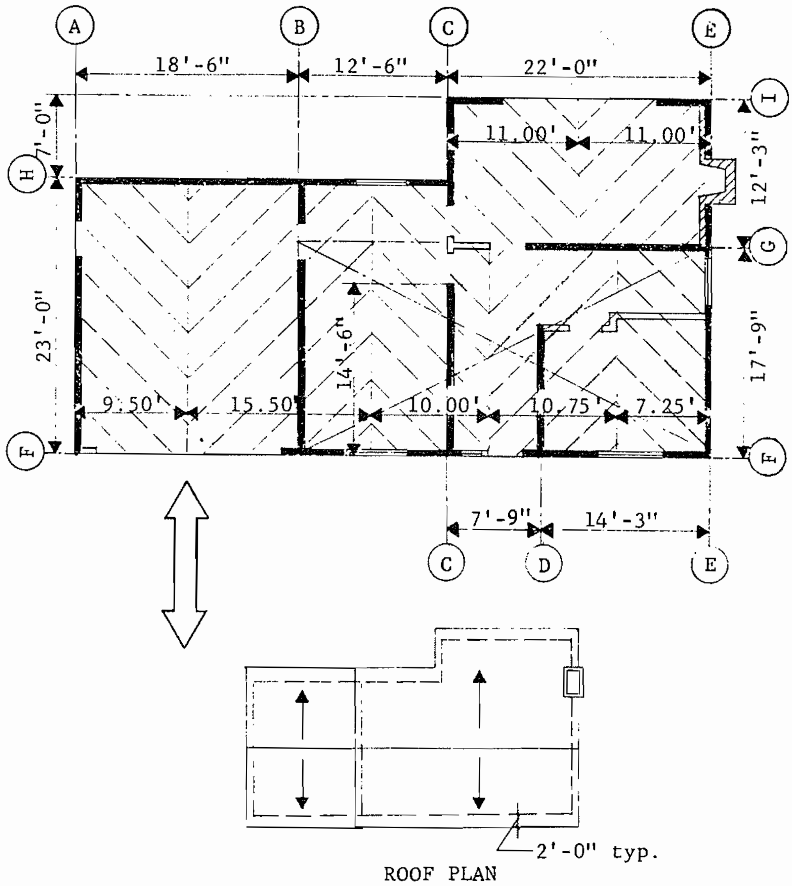


Figure B.8A

1. The diaphragm ratio of the second floor would be 34.50/17.75 = 1.94 if no interior walls were used. This is greater than 1.5:1: The wall on Line C must therefore be used in order to reduce the diaphragm ratio.

2. When the wall on Line C is used the diaphragm ratio between C and E becomes 21.75/17.75 = 1.22. The wall on Line D is 11.5/14.5 = .79 as long as the wall on Line C. Since this exceeds 0.75, in accordance with Table 3.1 this wall must also be used.

3. The load from the high roof should be distributed to the second floor walls as well as the

walls shown at the first floor above Line G. Although no second floor plan is shown, in all probability all roof load will be taken at Lines B and E with the load to Line C from the roof being treated as a wing. The tributary area plan shown is therefore only indicative of the load to the first floor walls above Line G.

4. Fireplace load will be taken by the wall on Line E.

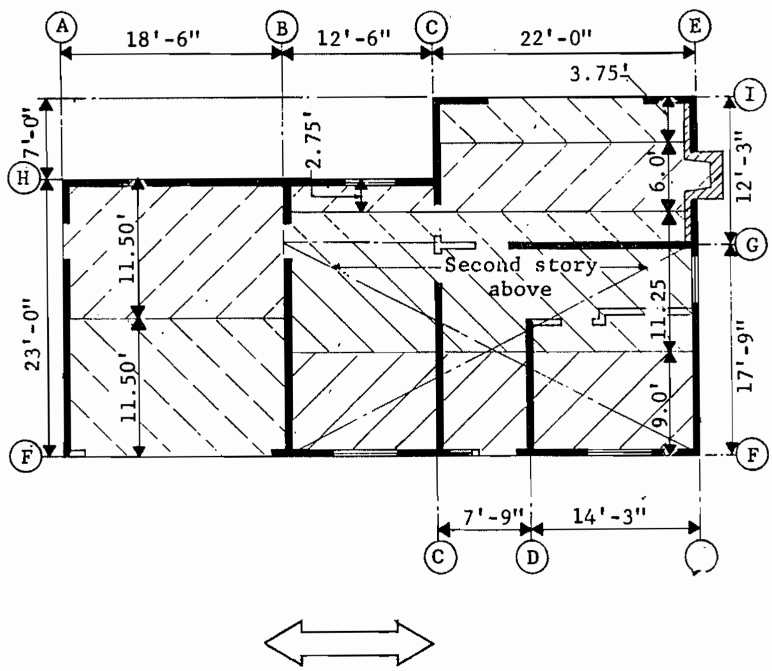


Figure B.8B

1. Since the plate line is continuous across the front of the house, there is no necessity for shear wall in the garage since there is ample shear wall along the front of the house (last sentence, Section 2.3D.)

2. The tributary area plan, in this particular instance, indicates the tributary areas for the roof as well as for the second floor.

3. The walls on Lines H and I should not be considered as acting together in accordance with Section 3.2B.

4. Fireplace load will be taken by the wall on Line H.

FIGURE B.9

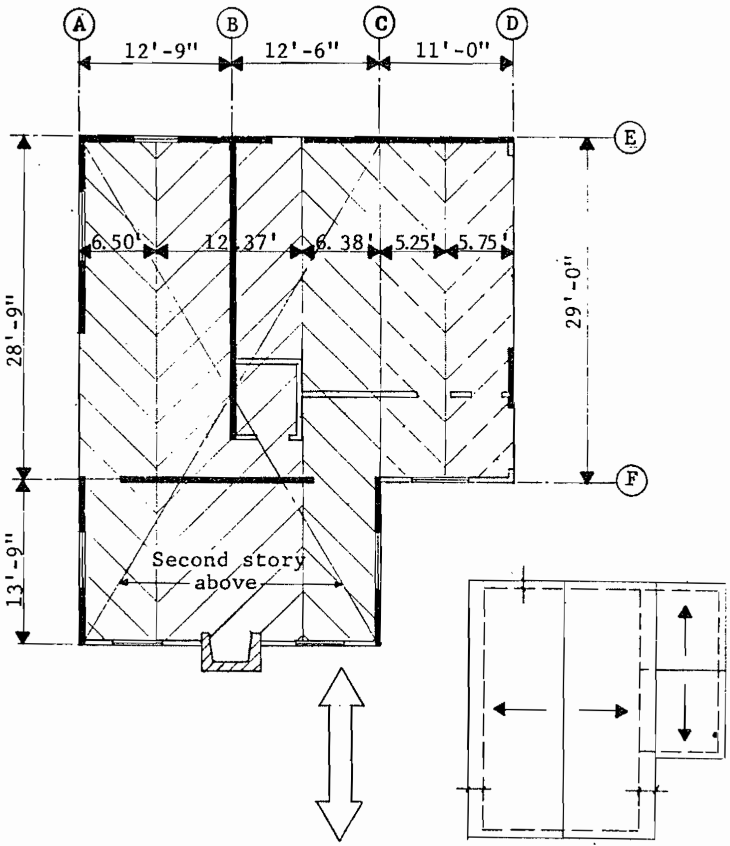
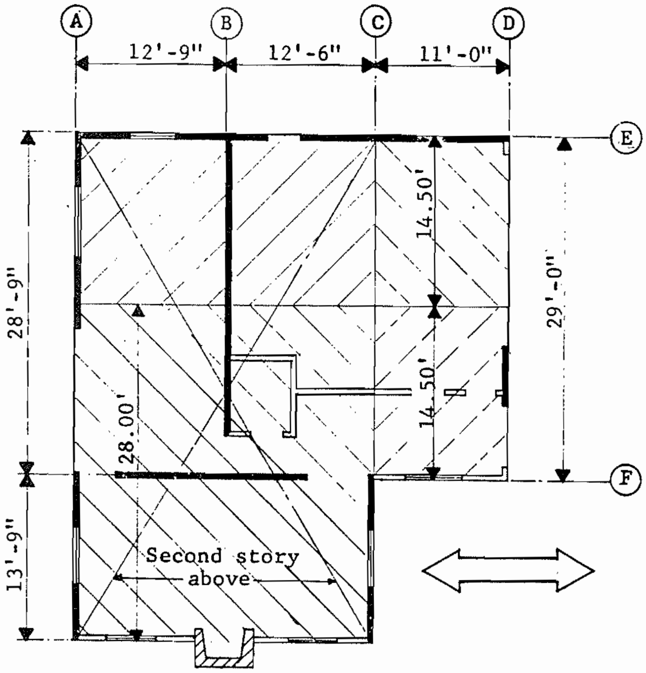


Figure B.9A

1. The multiplying factor for Line B is figured inthe same manner as shown for Model B (Figure 2.24B).

2. Obviously a beam will need to be provided across the garage on Line C in order to support the second floor wall above. This beam will act as a strut to carry load over to the walls on Line C. The walls on this line may prove to be insufficient for the considerable lateral load they are required to resist.

3. Fireplace load will be taken by the wall on Line B.



<u>**Figure B.9B**</u>

1. The walls on either side of the fireplace are too short to be considered as shear walls. It is therefore necessary to consider the diaphragm as a cantilever. It would be much more advisable to narrow down or relocate the windows in such a manner as to allow shear wall on either side of the fireplace.

2. Note that the top plates along Line F in all probability tie together the wall adjacent to Line D, and the wall between A and C.

3. The interior wall adjacent to the garage near Line F is not long enough to be required to be considered as shear wall. If the offset portion of wall were within three feet of this wall, the two sections together would need to be considered.

4. The same wall discussed above almost certainly extends to the garage roof as a division wall between the garage and the entry. Since the wall would be solid down to the door header, which would be below the height of the second floor, this wall could be utilized as a shear wall for the roof between Lines C and D and yet not be required to be considered as a shear wall for the two-story portion. This is not advisable. Either the wall should be designed for all load it might receive, or not considered at all.

5. Fireplace load will be taken by the wall on Line F.

FIGURE B.10

The roof plan for this residence indicates the roof over the one-story portion is extremely steep and, in fact, the ridge height is the same as that of the ridge over the two-story portion.

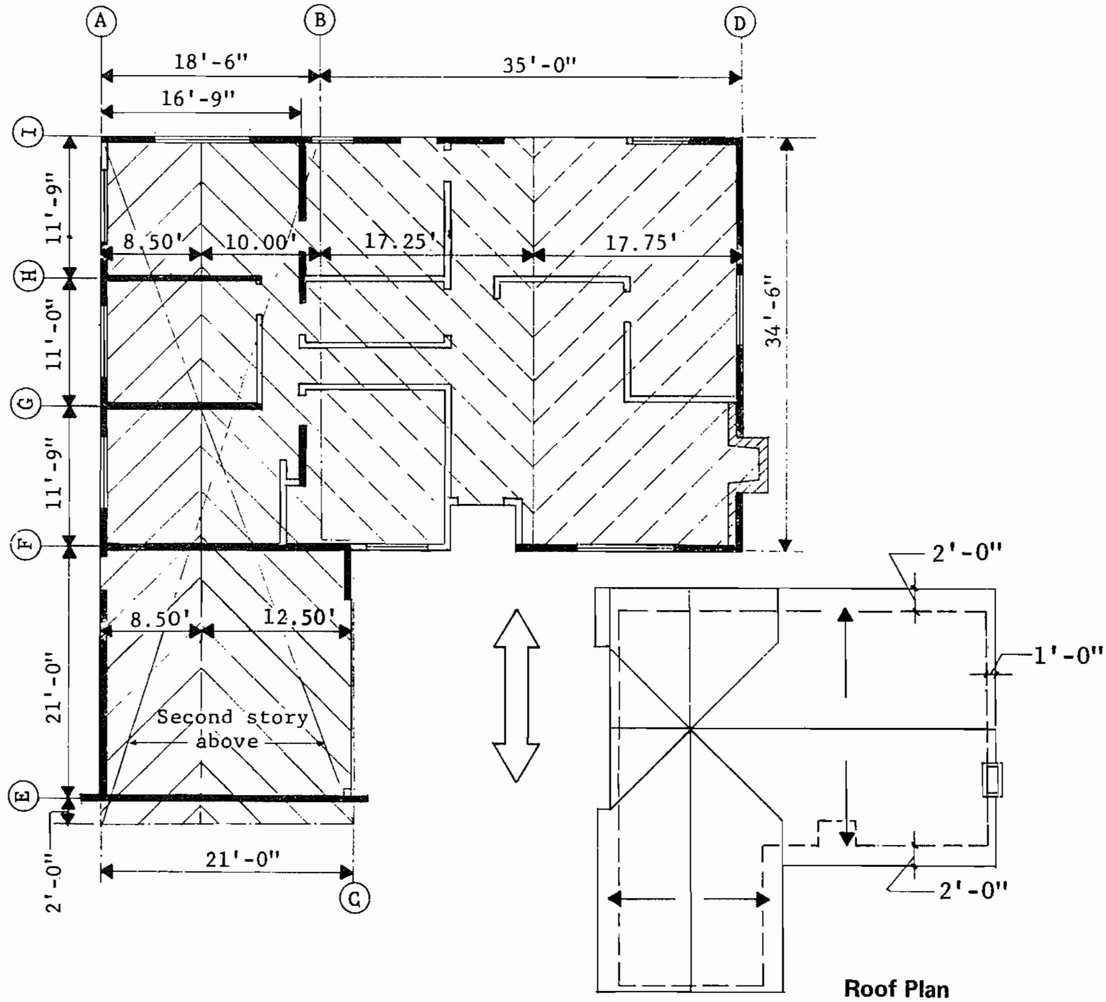


Figure B.10A

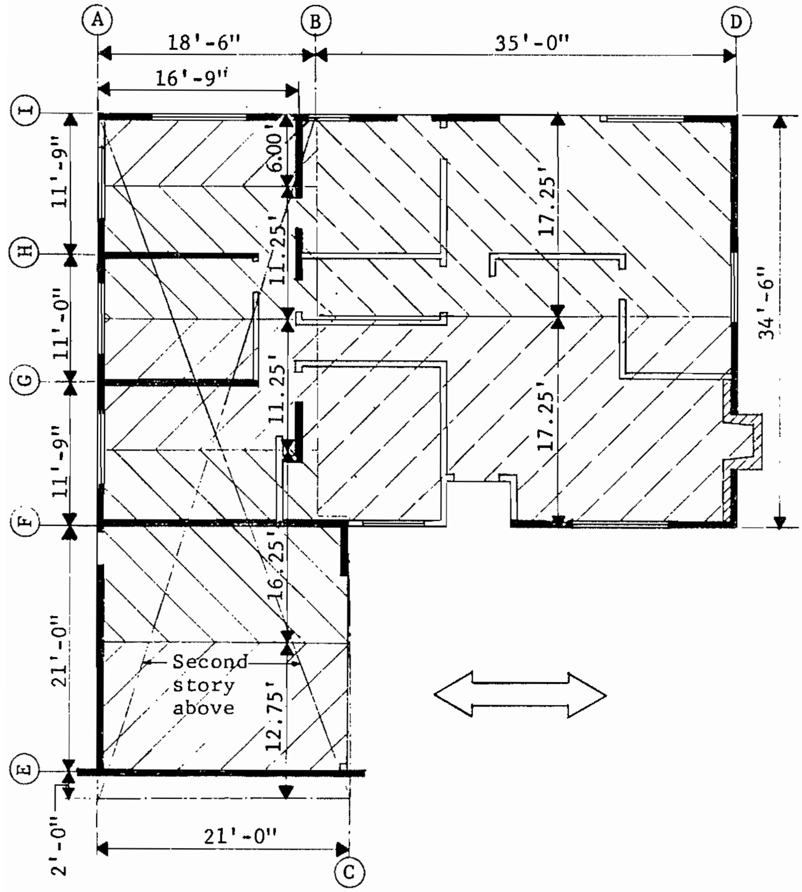
1. The wall to the left of Line B and the wall on Line C shall be considered as acting together.

2. The one-story roof shall be taken by the second story wall at Line B which in turn transfers its load into the second floor diaphragm.

3. The wall to the left of Line B is in accordance with Section 2.3D.

4. The width of the area for the wall to the left of Line B is greater than the width to Line A because of the cantilevered second floor diaphragm.

5. Fireplace load will be taken by the wall on Line D.



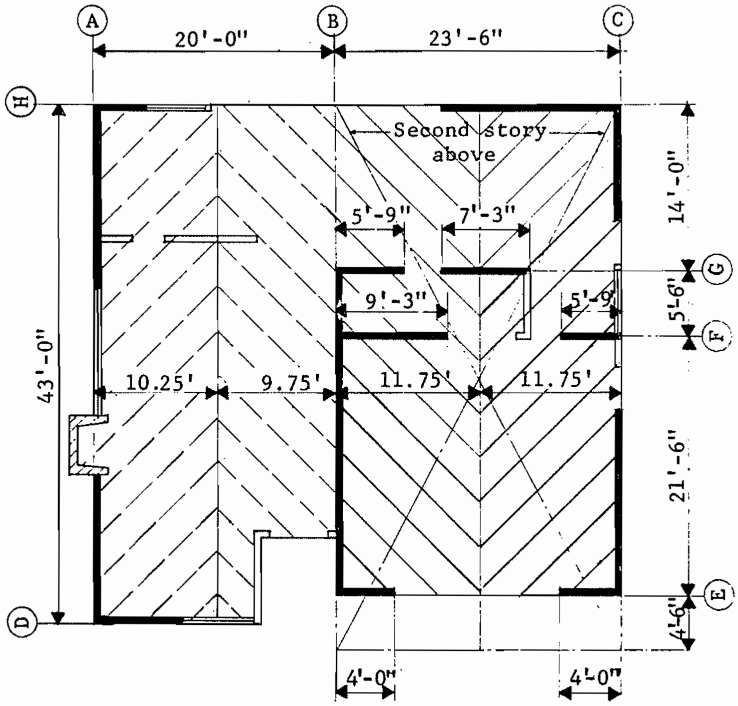
<u>**Figure B.10B**</u>

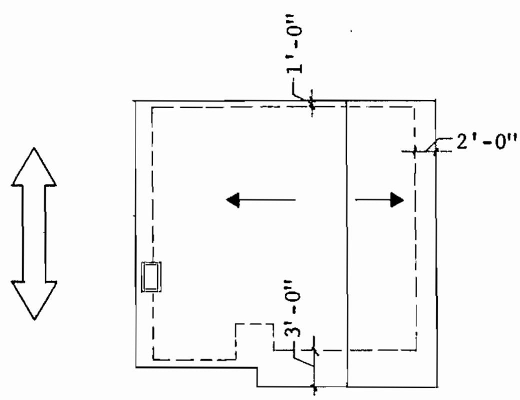
1. Since the low roof is not a portion of the second floor, the second floor diaphragm ratio between Lines F and I is 34.5/18.5 = 1.86 which is greater than 1.5:1. The interior walls at Lines G and H must both be used since they are the same length.

2. Fireplace load will be taken by the wall on Line F.

FIGURE B.11

This home was constructed with roof plans similar to example houses C and D. The roof plan shown is similar to example D.





ROOF PLAN

Figure B.11A

1. This split-level house is somewhat different from the configuration shown in the example homes, but its tributary areas in this direction are figured in exactly the same manner.

2. Fireplace load will be taken by Line A.

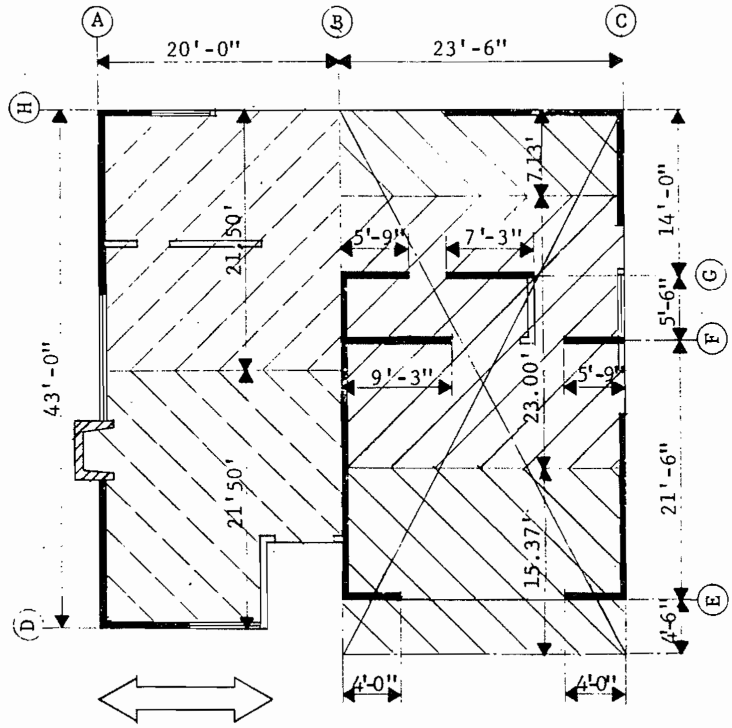


Figure B.11B

1. The walls at Lines F and G are considered to be acting together. Section 3.3B3(b) is therefore used as demonstrated by Figure 3.4. The adjusted width at Lines F and G is determined as follows:

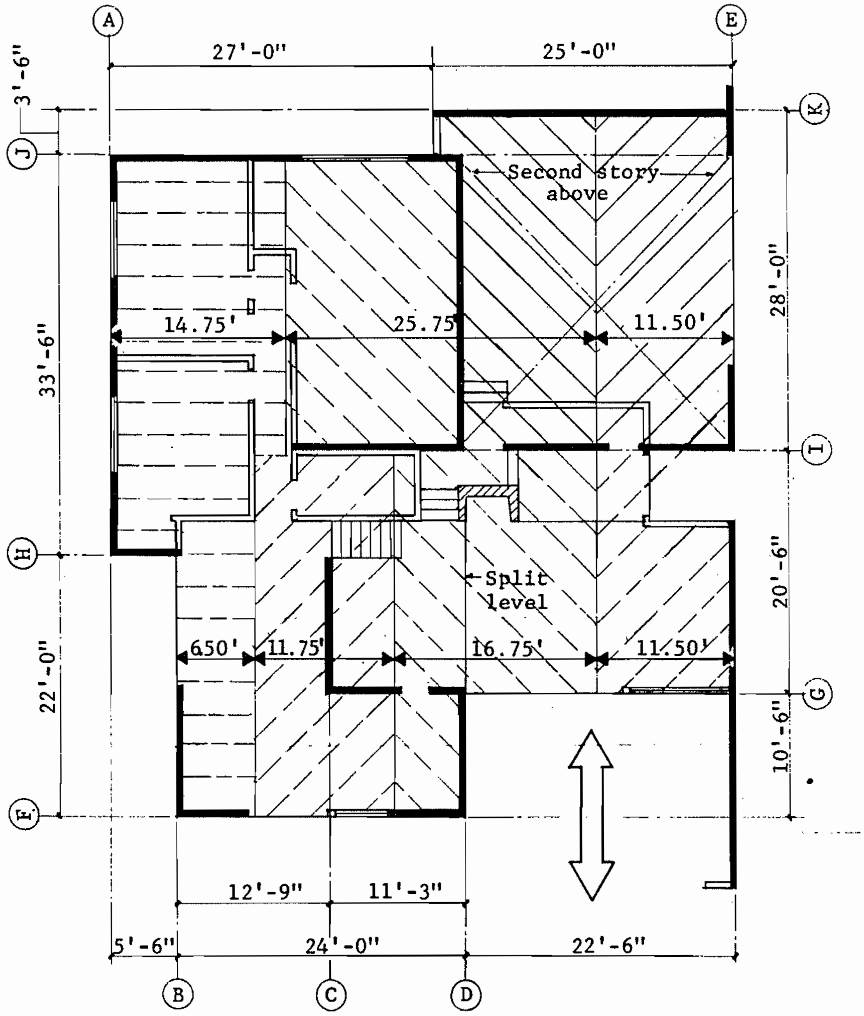
$$\frac{L_o}{L_i} = \frac{8.0}{28.0} = 0.286$$

Multiplying factor = 1.277 (from Figure 3.2)
Adjusted width = 1.277 x (23.00 - 5.5) + 5.5 = 22.35 + 5.5
= 27.85'

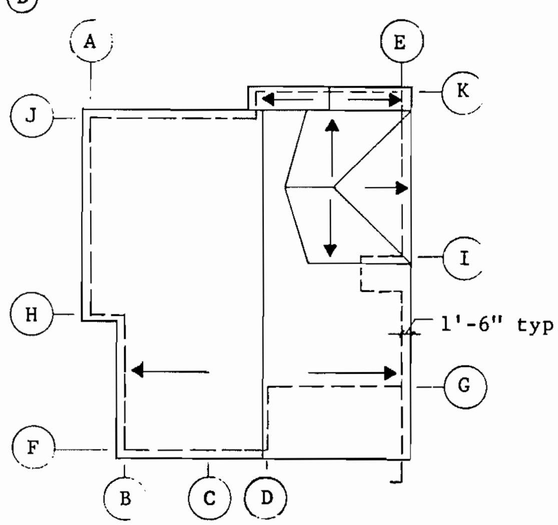
2. All of the area of the second floor cantilever is assigned to Line E per Design Section 3.3B6.

3. Fireplace load will be taken by the wall on Line D.

FIGURE B.12



This split-level home is considerably different from the models previously considered in that the only two story portion is over the garage, and the living-dining area contained between Lines G to I and Lines C to E contains a cathedral ceiling. The split in levels occurs at Line D. The one story roof is quite steep and the roof over the second floor dies into this steep roof.



Roof Plan

Figure B.12A

1. The walls on Lines A and B shall be considered as acting together.

2. The wall on Line C is considerably shorter than the walls on Line D and therefore the length of the tributary area is determined taking into consideration the general principle stated at the beginning of Design Section 3.3.

3. The two walls on Line D take almost one-half of the house. The segment of wall between I and J receives load from the second floor diaphragm as well as the roof. This segment should be extended to the roof it it is to act as indicated.

4. The two walls on Line D can be tied together by the ridge beam at the cathedral ceiling which will act as a strut to distribute roof load between them. The beam should be strapped to the blocking over the wall at each of its ends.

5. Although the wall on Line D between I and J is two stories in height, the wall between F and G is just as high with no second floor. The roof load should be distributed between the walls in proportion to their length as explained in Chapter II—4. In addition to the load from the roof, the wall from I to J only will be required to carry the second floor load.

6. The first story plate height on Line E from I to K is the same as the plate height for the roof from F to I and therefore these walls will act together. The portion of wall below Line F on line E may be considered as shear wall since it has a short section of wall at right angles to stiffen it. Normally it is not a good idea to consider walls projecting beyond the diaphragm as shear wall, since the wall is apt to buckle at the top.

7. Fireplace load will be taken by the wall on Line D.

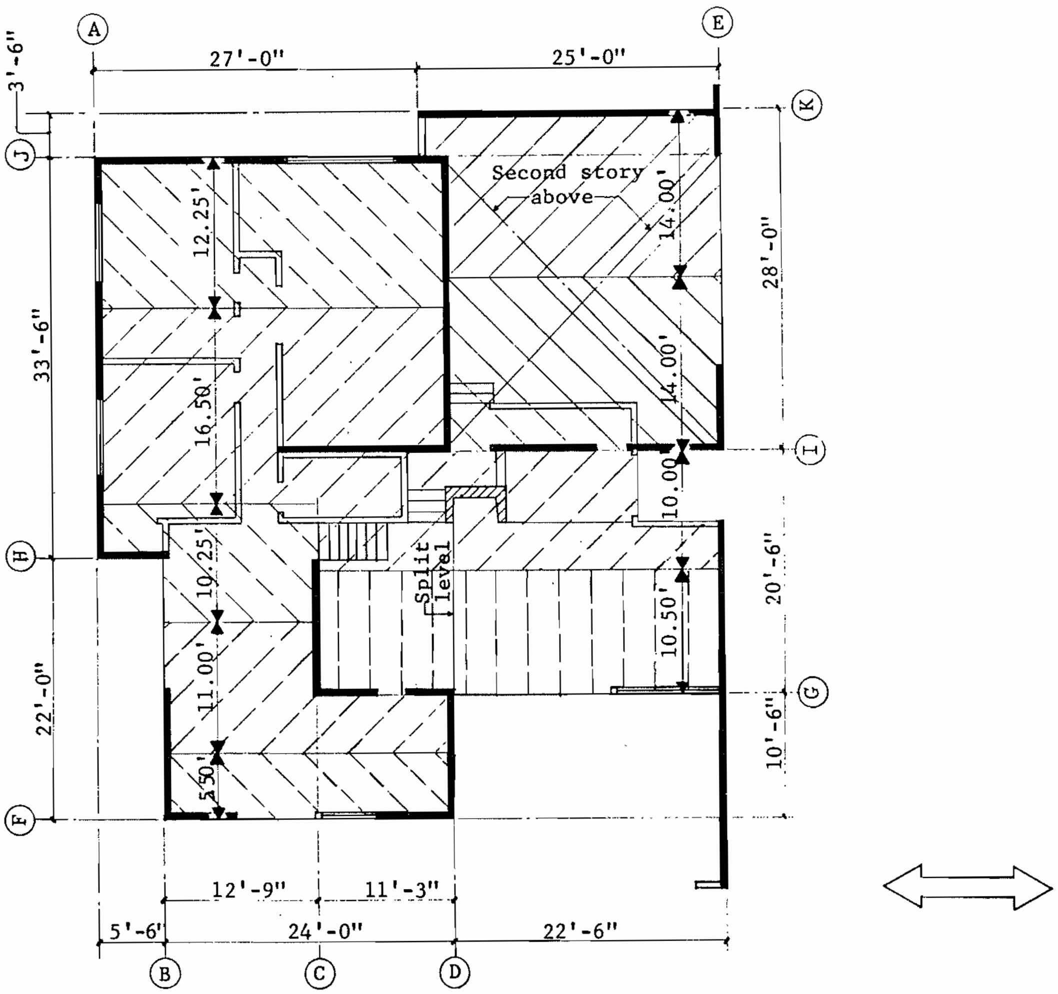


Figure B.12B

1. Note that shear walls have been placed all around the area containing the cathedral ceiling.

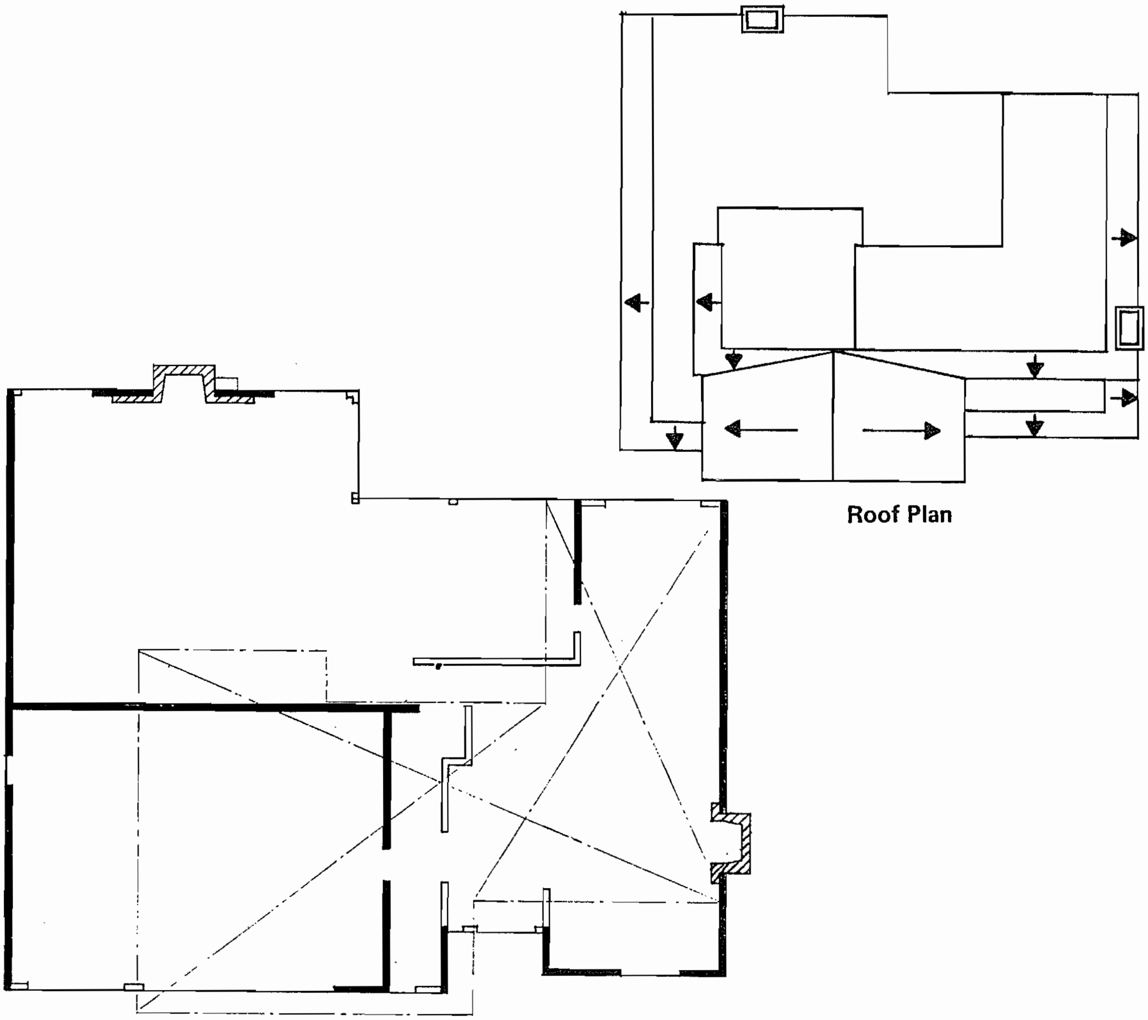
2. The walls on Lines H and I present some complex problems. The wall on Line I between D and E is in one portion of the split-level structure and should be designed for loads tributary to it from the two level portion, but in addition receives load together with the remainder of the wall on Line I from the cathedral ceiling. The portion of the wall extending to the cathedral ceiling above Line D is obviously quite high but only receives load from the roof. The wall on Line H is 9'-0" from I and could therefore be considered as acting together with Line I. This condition demonstrates most clearly where judgment enters into the division of tributary areas. Since the wall on Line H is so much lower than the wall in the one-story portion on Line I, the tributary areas have been figured separately.

   In addition, the entire wall on Line I is considered as taking load from the cathedral ceiling and the one story portion of the house above Line D. The second floor load should be distributed only to that portion of the wall on Line I adjacent to the two-story portion, since no plates extend from the two-story wall to the portion above Line D. With two-story wall and variable height wall mixed together with a plan which is a mixture of one and two story combination and split-level, no other recourse is available than the use of judgment in determining the areas which should be tributary to each wall.

3. The roof of the house can be taken by the shear walls on Lines J and K acting together, while the two-story portion should be resisted by the wall on Line K unless a strut is provided to J. Note that in order to transfer the two-story load to Line K, it is necessary to extend the second floor diaphragm to Line K. The roof between J and K can then be false-framed over the extended diaphragm.

4. The guidelines for determining tributary areas cannot be completely followed because of the complex nature of this house. When this occurs it is usually a good indication that engineering would be desirable.

FIGURE B.13



The roof plan of this house is as structurally complicated as it appears. The high flat roof over the second floor is at two levels reflecting the vertical offset of the second floor levels below. The low roof is also essentially flat. The narrow strips of sloping roofs are false mansards. The only portion of genuinely sloping roof is the small section over the second floor at the front.

In addition to being at two levels, the second floor diaphragm is unsupported where it reaches the rear of the house, just below the front fireplace and at the far left hand side, all as shown on Figure B.13.

The design methods set forth herein are not intended to reach the degree of engineering sophistication required to provide a seismic analysis of this complex residence. When attempting to design a structure of this type, engineering assistance should be obtained for both vertical and lateral load analyses.

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