ARMY TM 5-809-2
AIR FORCE AFM 88-3, CHAP. 2

TECHNICAL MANUAL

STRUCTURAL DESIGN CRITERIA FOR BUILDINGS

APPROVED FOR PUBLIC RELEASE; DISTRIBUTION IS UNLIMITED

DEPARTMENTS OF THE ARMY AND THE AIR FORCE

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REPRODUCTION AUTHORIZATION/RESTRICTIONS

TECHNICAL MANUAL
No. 5-809-2
AIR FORCE MANUAL
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HEADQUARTERS
DEPARTMENTS OF THE ARMY
AND THE AIR FORCE
Washington, DC 20 May 1992

STRUCTURAL DESIGN CRITERIA FOR BUILDINGS

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* This manual supersedes TM 5-805-3, dated 15 July 1981; TM 5-809-2/AFM 88-3, Chap. 2, dated 31 August 1984; TM 5-809-4/AFM 88-3, Chap. 4, dated 15 February 1983; TM 5-809-5/AFM 88-3, Chap. 5, dated 1 June 1983, and TM 5-809-8/AFM 88-3, Chap. 8, dated 5 March 1984

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CHAPTER 1

GENERAL

1-1. Purpose

This manual furnishes structural design criteria for buildings.

1-2. Scope

Materials covered in this manual include concrete, steel, wood, and aluminum. Building components include metal roofing and siding. This manual does not cover specialized construction such as patented curtain wall systems which may have applicability in certain situations. Masonry design criteria are covered in TM 5-809-3/AFM 88-3, Chapter 3.

1-3. References

Appendix A contains a list of references used in this document.

1-4. Alternative designs

Deviation from these criteria, where a valid need exists and where an alternative solution is more desirable with an equivalent or better result based upon sound engineering principles, may be accepted subject to approval by the appropriate headquarters.

1-5. Design loads

a. General. Design loads will conform to TM 5-809-1/AFM 88-3, Chapter 1. Seismic design loads will conform to TM 5-809-10/AFM 88-3, Chapter 13; TM 5-809-10-1/AFM 88-3, Chapter 13, Section A; or TM 5-809-10-2/AFM 88-3, Chapter 13, Section B. Loadings not covered by these manuals will be obtained from available technical literature or will be carefully formulated considering accepted practice for the location.

b. Roofs. Design of roofs will include consideration of the effects of the maximum allowable deflection of the roof under maximum design loads. For consideration of typhoons and hurricanes, refer to TM 5-809-11/AFM 88-3, Chapter 14.

c. Deflection. The deflection of all structures and components will be in accordance with the Uniform Building Code. For construction supporting masonry walls or partitions, the maximum deflection due to dead and live loads, including long-term effects, will not exceed L/480. More restrictive deflection criteria will be considered for special situations, e.g., crane or monorail beams, fragile building components, etc., in accordance with accepted industry practice or as set forth elsewhere in this manual.

1-6. Special designs

a. Design. Prior approval for special designs will be obtained from the appropriate headquarters, HQUSACE (CEMP-ET) WASH, DC 20314-1000 for Army projects or HQ, USAF/LEEDF, Bolling AFB, WASH, DC 20332 for Air Force projects.

(1) Where the use of materials or systems for structural framing which are not covered by this document or the documents referenced herein appears necessary, advantageous, and economical.

(2) Where the use of unique structural systems such as long-span or three-dimensional structural framing systems is desirable even though they may require special design; controls; and research efforts into their construction and performance history. The use of a unique structural system and/or material is discouraged.

b. Procedures. The request for approval will contain a complete statement of the reasons for using such a system, including competitive costs, proposed special criteria and controls as applicable, performance history or tests (if available), the use of a recognized structural consultant for the design of the unusual structures, and other pertinent data. The approval will apply to the specific project for which the special design use was requested and will not apply to other projects involving a similar application. Prior approval is required under the following circumstances.

(1) Where the use of materials or systems for structural framing which are not covered by this document or the documents referenced herein appear necessary, advantageous, and economical.

(2) Where the use of unique structural systems, such as long-span or three-dimensional structural framing systems is desirable even though they may require special design; controls; and research efforts into their construction and performance history.

c. Fabric structures. Fabric structures are those which utilize an enclosure fabric acting in tension as a structural element. The use of fabric structures is particularly applicable to temporary construction, to situations requiring minimum structure weight, or to conditions requiring large, column-free spaces. Fabric structures are typically either air-supported or supported by a system of cables which in turn are supported by other structural elements acting in compression. For additional guidance on the design of fabric structures, see Tension Structures Behavior and Analysis by Leonard and refer to literature available from the major manufacturers of structural fabrics and air-supported structures. Use of fabric structures is subject to prior approval as set forth above except when they are used as temporary enclosures.

1-7. Overseas construction

Where local materials of grades other than those referenced herein are to be used, the working or yield strength stresses and details of construction will be modified as required by the structural property characteristics of the local material. The material, as far as practicable, will be of equivalent or better grade than the comparable grades referenced herein.

1-8. Service life

Service life of various buildings and facilities is defined as follows:

a. Permanent Construction. Permanent construction will be designed and constructed to serve a life expectancy of 25 years or more, will be energy efficient, and will include finishes, materials, and systems selected for low maintenance and low life-cycle costs.

b. Limited life structures. Limited life structures include both semi-permanent and temporary construction as described below.

(1) Semi-permanent construction will be designed and constructed to serve a life expectancy of more than 5 years but less than 25 years (generally a 15-year service life), will be energy efficient, and will include finishes, materials, and systems selected for a moderate degree of maintenance using the life-cycle approach.

(2) Temporary construction will be designed and constructed to serve a life expectancy of 5 years or less, will use low-cost construction, and will include finishes, materials, and systems selected with maintenance factors being a secondary consideration.

1-9. Stability

Unless noted otherwise, when discussed in this manual, stability relates to sliding, overturning, buoyancy, and other sources of gross displacement and not to stability as related to buckling. A structure or any of its elements will be designed to provide a minimum safety factor of 1.5 against failure by sliding, overturning, uplifting, or buoyancy. This required degree of stability will be provided solely by the dead load plus any permanent anchorage.

a. Designs will be such that the serviceability of the structure is maintained under all probable loadings to which the structure will be subjected on a frequency of once every ten years (or longer) except that for a critical structure, such as a hospital, serviceability will be maintained throughout its design life.

b. Designs will be such that the integrity of the structure is assured under all conditions of loading up to and including the maximum combinations applicable to the structure's design life.

CHAPTER 2
CONCRETE

2-1. Introduction

This chapter prescribes criteria for the design of buildings using cast-in-place or precast construction with plain, reinforced, or prestressed concrete.

2-2. Basis for design

a. General. Design of concrete will be in accordance with provisions of the American Concrete Institute (ACI). In executing designs in accordance with ACI 318, cognizance will be given to ACI 318R; Portland Cement Association (PCA) Notes on ACI 318 Building Code Requirements for Reinforced Concrete with Design Applications; ACI 301; ACI 318.1; and to ACI standards and special publications referenced in those documents.

b. Buildings.

(1) Reinforced concrete. ACI 318 will apply. For guidance in the use of this ACI standard and its design applications, see PCA Notes on ACI 318 Building Code Requirements for Reinforced Concrete with Design Applications.

(2) Plain concrete. ACI 318.1 will apply.

(3) Concrete cover for severe exposure conditions. See paragraph 2-15.

(4) Concrete footings bearing on rock (or other material as hard). Design on basis of distribution of load within the footing at an angle of 30 degrees from the vertical without the development of bending.

(5) Special requirements for seismic areas. See TM 5-809-10/AFM 88-3, Chapter 13.

(6) Slab systems. See paragraph 2-5.

(7) Fire endurance. ACI 216R will apply.

(8) Durability. ACI 201.2R will apply.

2-3. Design strengths

The design strengths for reinforced concrete in buildings will conform to ACI 318 for reinforced concrete and to ACI 318.1 for plain concrete. Design strengths for seismic design will conform to TM 5-809-10/AFM 88-3, Chapter 13. Concrete strengths for various applications and various exposures are listed in table 2-1. Use of high strength concrete will be in accordance with ACI 363R and SP-87.

2-4. Design choices

The selection of structural concrete framing system, strength of concrete and reinforcement, conventional versus lightweight concrete, conventional versus prestressed design, and cast-in-place versus precast construction will be based on economic and functional considerations taking into account the specific type and size of structure, architectural features or special performance requirements, seismic exposure, construction cost factors for the building site, and the availability of materials and labor. For further discussion of considerations in selecting appropriate composition and properties for concrete, see TM 5-805-1/AFM 88-3, Chapter 6 and PCA Design and Control of Concrete Mixtures.

2-5. Slab and framing systems

a. Slab systems. Generally floors and roof slabs will be one-way solid slabs, two-way solid slabs, one-way joists, two-way ribbed (waffle) slabs, flat slabs, or flat plates.

b. Framing systems. Framing will be rigid frame or bearing and/or shear walls.

2-6. Shrinkage and temperature reinforcement

The minimum shrinkage and temperature reinforcement provisions of ACI 318 will be used.

2-7. Joints

Joints are used in concrete construction for a number of reasons. The following are basic requirements for the more common types. Refer to PCA Design and Control of Concrete Mixtures and PCA Building Movements and Joints for additional information.

a. Expansion joints. The applicable criteria of the National Academy of Sciences-National Research Council, Building Research Advisory Board, Federal Construction Council Technical Report No. 65 will apply, except as modified by these standards. Where criteria are lacking the following will serve as guidelines subject to modification based on consideration of local experience, changes in atmospheric moisture content, anticipated shrinkage, shape of structure, amount and type of reinforcing, conditions of restraint, and other sources of secondary and parasitic stresses.

(1) Where the temperature differential (TD), defined as the greater of the differences between the annual mean air temperature and the highest and lowest air temperature be expected, is not greater than 70 degrees F and no excessive change in atmospheric moisture is anticipated, expansion joints should be spaced so straight lengths of building measure no more than 300 feet between joints.

(2) Where the TD is greater than 70 degrees F, or where excessive change in atmospheric moisture is likely (as may be expected in parts of the tropics), expansion joints should be spaced so straight lengths of building measure no more than 200 feet between joints.

(3) An expansion joint is usually required between djoining building areas which are different in shape, or between areas where different rates of building settlement are anticipated.

(4) Joints should be located at junctions in L-, T-, or U-shaped buildings and at points where the building is weakened by large openings in the floor construction such as light wells, stairs, and elevators.

(5) Expansion joints in retaining walls or other exposed concrete (where temperature change of the concrete mass is not modified by proximity of temperature-controlled space) should be located no more than 100 feet apart and at 40-foot spacing, if feasible.

b. Control joints. Provide control joints in walls to limit and conceal cracks as follows:

(1) In walls with openings, space control joints at 20-foot intervals; in walls with infrequent openings, space at 25-foot intervals.

(2) Provide a joint within 10 or 15 feet of a corner.

Table 2-1. Concrete strengths.

Usage	*Minimum Strength (psi)*
Concrete fills.	2000
Encasements for utility lines and ducts.	2500
Concrete exposed to frost action where 2000 psi concrete would be used.	3500 to 4000
Foundation walls and footings.	3000
Cast-in-place concrete piles for shore use.	3000
Retaining walls subject to ordinary exposure conditions.	3000
Slabs on grade.	3000
Reinforced concrete buildings and similar structures.	3000
Precast members (including architectural and structural members and piles).	4000 to 5000
Walls or floors subjected to severe exposure. (Severe exposure includes extreme heat or cold and exposure to deicing or other aggressive chemicals.)	3000 to 4000
Concrete deposited under water (tremie concrete).	3000 to 4000
Columns in multistory buildings carrying heavy loads.	4000 to 5000
Reinforced members in buildings and similar structures where smaller sections are necessary for clearance or where higher working stresses are economical, such as columns and long, heavy girders.	4000 to 5000
Reinforced concrete in contact with sea-water, alkaline soils or waters, or other destructive agents.	4000 to 5000
Prestressed concrete construction	5000

(3) Where steel columns are embedded in the walls, provide joints in the plane of the columns.

(4) If the columns are more than 25 feet apart, provide intermediate joints.

(5) For retaining walls or other concrete walls where neither face is in a temperature-controlled space, provide joints every 20 to 25 feet.

c. Construction joints. Construction joints are used to allow concrete placement of separate construction elements at different times, e.g., between columns and beams, footings and pedestals, etc. Construction joints will be made with tie bars, dowels, or keys to provide shear transfer. The location and details of critical construction joints will be shown on the drawings and, to the extent practicable, will coincide with the location of expansion or control joints. The location of other construction joints need not be shown. Cautionary and advisory notes regarding acceptable joint locations will be included on the drawings.

d. Sealing joints. Exterior expansion, control, and construction joints should be sealed against moisture penetration using methods such as waterstops or sealants as appropriate for the prevailing conditions.

2-8. Deflection and crack control

The effects of deflection, creep, shrinkage, temperature contraction and expansion, and the need for vibration isolation in concrete construction will be considered. Appropriate allowances for these factors will be included in the design; location and details or provisions for required control (weakened-plane) joints, expansion joints, isolation joints, and location of water stops will be shown on the drawings. Specific locations for horizontal and vertical construction joints will be shown where such joints are critical with respect to design considerations, e.g., configuration of the structural concrete, effects of construction joints on structural strength and shrinkage cracking, and appearance of construction joint lines on exposed concrete. Continuous wall footings will be constructed with the minimum practicable number of construction joints. Excessive cracking can occur when concrete slabs are cast continuously over supports. Therefore, additional reinforcement is required at these locations to prevent large crack development. This type of cracking has occurred above the ends of simply supported steel joists with concrete placed continuously on metal deck. Where reinforced concrete foundation walls support masonry, crack control measures will be designed to be compatible with crack control measures in the masonry. All crack control joints in the foundation wall will be carried upward into masonry crack control joints.

2-9. Climatic influences

Extremes in climatic conditions (freezing temperatures, tropical environments, etc.) should be considered in determining concrete composition. For further discussion of relevant factors, see TM 5-805-1/AFM 88-3, Chapter 6.

2-10. Special considerations

a. Shear transfer.

(1) Analysis. The analysis of shear transfer will be in accordance with provisions of ACI 318. Special attention will be given to transfer of shear at locations such as shear heads, bases of walls, brackets and corbels.

(2) Compatibility. The combined action of flexible and rigid shear connectors will not be considered as providing simultaneous shear transfer. Rigid shear connectors include roughened surfaces and structural shapes. Flexible connectors include bolts, stirrups, dowel bars, and ties.

b. Concrete anchorage. Criteria on the various anchorage system alternatives and their usages will be in accordance with ACI SP 103. Allowable shear and tension values for anchor bolts are listed in table 2-2.

2-11. Structural slabs

The following special requirements for structural slabs will be considered:

a. Slabs supporting masonry walls or partitions. The total deflection, including long-term effect will be limited as set forth in chapter 1.

b. Cast-inplace roof slap and floor decks. For roof slabs and floor decks placed monolithically with deep girders or beams, considerations will be given to providing additional reinforcing bars in mid-depth of slabs at all column heads. Such reinforcing will be placed prepndicular to the diagonals of the slabs and will extend over or into the beam or girders at the column heads. A minimum of four No. 4 bars spaced at 6 inches on center will be arranged diagonally at each slab corner. Size and number of bars may be increased depending on the manitude of the load. (These added bars are to preclude diagonal radial slab cracks apparently caused by the slab acting as part of tension flanges of the intersecting beam or girders and by shrinkage factors. These cracks tend to occur when slabs are relatively thin with beams or girders spanning 30 feet or more from column to column).

c. Prestressed slabs. Basis for design of prestressed concrete slabs will be in accordance with ACI 318, Chapter 18.

d. Post-tensioned slabs. Design of post-tensioned slabs will conform to the following Post-Tensioning Institute (PTI) publications: Design and Construction of Post-Tensioned Slabs-on-Ground. Post-Tensioned Commercial and Industrial Floors.

2-12. Slabs on grade

a. Applicable conditions. Criteria presented in this manual apply to lightly loaded slabs on grade, normal

*Table 2-2. Allowable shear and tension on bolts.**

Bolt Diameter (In.)	Minimum Embedment (In.)	Minimum Concrete Strength 2000 psi (Shear)	3000 psi (Shear)	2000 to 5000 psi (Tension)
1/4**	2-1/2	500	500	200
3/8**	3	1,100	1,100	500
1/2**	4	2,000	2,000	950
5/8**	4	2,750	3,000	1,500
3/4	5	2,940	3,560	2,250
7/8	6	3,580	4,150	3,200
1	7	3,580	4,150	3,200
1	8	3,580	4,500	3,200
1-1/4	9	3,580	5,300	3,200

*Values are for natural stone aggregate concrete and bolts of at least ASTM A 307 quality. Bolts shall have a standard bolt head or an equal deformity in the embedded portion. Values are based upon a bolt spacing of 12 diameters with a minimum edge distance of 6 diameters. Spacing and edge distance may be reduced 50 percent with an equal reduction in value. Use linear interpolation for intermediate spacings and edge distance.

** For most applications, anchor bolts less than 3/4 inch diameter should not be used because it is possible to strip the threads on these size bolts with a hand wrench, and it is difficult to replace anchor bolts embedded in concrete.

subgrade conditions, and average conditions relative to concrete placement, curing, and shinkage. Lightly loaded slabs on grade are those which are subjected to stationary live loads of not more than 400 pounds per square foot, stationary concentrated line (wall) loads of not more than 600 pounds per foot, or vehicle axile loads of not more than 5,000 pounds. Normal subgrade conditions are characterized by the absence of the following: conditions conducive to frost penetration, potential for significant volume change due to change in moisture content, and expansive soils; and by the presence of soils classified, according to American Society for Testing and Materials (ASTM) D 2487, as either Class ML, any of the S or G groups, or Class CH, MH, or CL having a modulus of subgrade reaction (k) of 100 pounds per cubic inch or greater. A tabulation of maximum allowable uniform live load for various thicknesses is presented in this paragraph. Refer to appendix B and appendix C for design of lightly loaded slabs on grade subject to concentrated or vehicular loads. Refer to TM 5-809-12/AFM 88-3, Chapter 15 for design of slabs on grade subjected to heavy loads.

b. General. A 6 mil (minimum) thick vapor barrier will be placed under the slab on grade and over the capillary water barrier. The effects of a vapor barrier on potential slab curling, bleeding, etc., will be carefully considered when establishing slab concrete mix, curing, finishing, and other requirements. The capillary water barrier will be below all HVAC ducts. Slabs will not bear directly on footings or pedestals; at least 6 inches of granular fill cushion or capillary water barrier will be provided between the slab and the concrete below. Slabs on grade in expansive soil areas require special consideration; mat foundations with or without ribs may be required. In such instances, texts on soil stabilization should be consulted, and consideration should be given to a post-tensioned foundation slab. For a discussion of this slab type and its design, see PTI Design and Construction of Post-Tensioned Slabs-on-Ground.

c. Thickness. Slabs will ordinarily have a minimum concrete compressive strength of 3,000 psi and a minimum thickness of 4 inches. The following thicknesses for maximum uniform design live loads will be used provided the modulus of subgrade reaction (k) is at least 100 pounds per cubic inch.

Thickness of Slab (inches)	Maximum Uniform Design Live Load (pfs)
4	150
5	250
6	400

With 4-inch thick slabs, maximum partition loads up to 400 pounds per linear foot will be permitted. When partitions are located at the free edge of a 4-inch slab, however, a maximum partition load of 300 pounds per linear foot will be permitted provided the edges are thickened in accordance with figure C-2 in appendix C. Where partition loads exceed these permitted maximums, a thickened slab designed in accordance with appendix C or a separate continuous wall footing will be used.

d. Crack control. Slabs will be placed in checker-board or lane fashion. The lane method is the preferred method because the checker-board method is more expensive and results in slabs having low surface tolerances. The area of sections bound by crack control joints will not exceed 625 square feet, and distance between crack control joints will not exceed 25 feet for slabs. The length/width ratio of panels bounded by joints will be as near 1.0 as possible and will not exceed 1.25. In localities where extreme conditions of heat or dryness tend to produce excessive shrinkage, the maximum area and joint spacing values should be decreased. Crack control joints may be construction joints. Joints in the vicinity of column pedestals will be placed at column center-line, with diamond shape isolation joints provided at columns or square shape isolation joints provided at column pedestals. When thickened slabs are used under column bases or partitions, joints should be offset from the thickened areas. Corners of isolation joints will meet at a common point with other joints so far as practicable. Where discontinuous joints, i.e., joints which are not continuous across their perpendicular joints (see figure 2-1), cannot be avoided, two No. 4 bars, 4 feet long, will be placed parallel to the edge opposite the end of the discontinuous joint. Bars will be at mid-depth and 4 inches apart starting 2 inches from edge of slab. Except for openings of less than 12 by 12 inches, corners of openings and reentrant corners in slabs will be reinforced with two No. 4 bars, 4 feet long, placed diagonally to the corner.

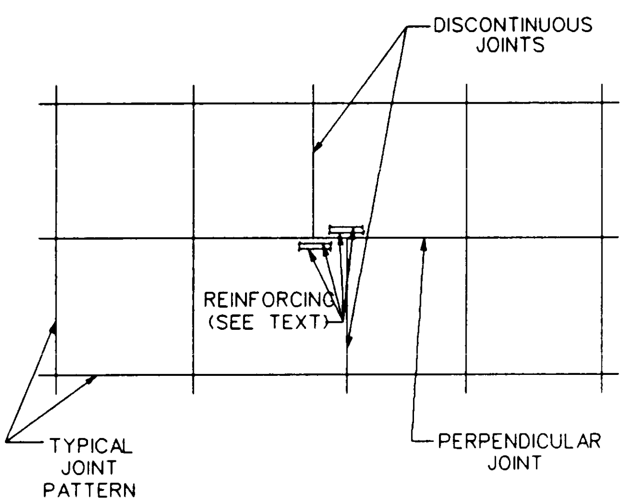


Figure 2-1. Discontinuous Joints.

e. Wire mesh reinforcement. Wire reinforcement, when used, will be continuous through slab construction joints (unless they are to serve as control or expansion joints) and partially interrupted (every other wire) within 2 inches of each side of slab control joints. Slabs will contain about 0.1 percent welded wire mesh reinforcement in each direction placed 1-1/2 inches from the top of slab. For example:

Slab Thickness	Wire Mesh Reinforcement
4-inch	6x6 -W2.0 xW2.0
5-inch	6x6 -W2.9 x W2.9
6-inch	6x6 -W4.0 x W4.0

Wire mesh reinforcement is not required provided that all of the following conditions are satisfied.

(1) Control joints in each direction are spaced no further apart than the following:

(a) For concrete using less than 3/4 inch maximum size coarse aggregate, 24 times the slab thickness.

(b) For concrete using greater than 3/4 inch maximum size aggregate, 30 times the slab thickness.

(c) For low-slump concrete (less than 4 inches), 36 times the slab thickness.

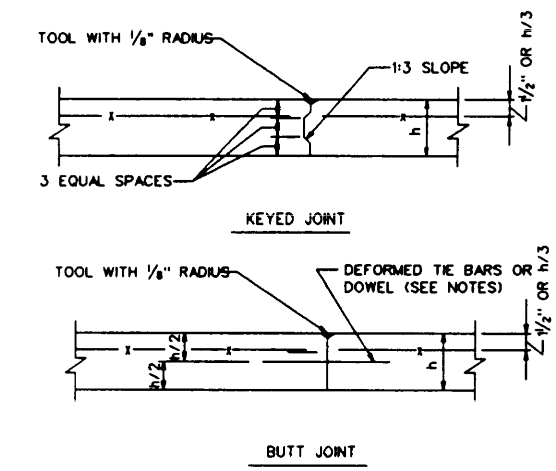
(2) The slab is constructed of concrete with a minimum f'c = 4,000 pounds per square inch.

(3) The slab is placed on subgrade where the subgrade has been graded to a tolerance of plus or minus 1/2 inch, where there is a high degree of confidence in the uniformity of compaction of the subgrade, and where the compacted subgrade has a minimum k (modulus of subgrade reaction) of 100 pounds per cubic inch.

f. Construction joints. Construction joints are used to allow separate concrete placement at different times (i.e., when concrete placement is stopped or delayed). Construction joints will be made with tie bars, dowels, or keys to provide shear transfer . Formed keyed joints will only be used in slabs having a thickness of 6 inches or more. Preformed keys left in place may be used for 4-inch and thicker slabs. The key will be centered on the depth of the slab with the base of the male portion about one-third the depth of the slab. Location and details of construction joints will be shown on drawings. Details of construction joints are shown in figure 2-2.

g. Expansion joints. Expansion joints will be designed to permit horizontal movement. Expansion joints will be made with plane face, 3/8-inch or more thick filler, and 3/4-inch smooth dowels 16 inches long, at 16-inch spacing in 4- and 5-inch slabs, and 12-inch spacing in 6-inch slabs. Half of the dowel length will be greased. Location and details of expansion joints will be shown on drawings.

h. Control joints. Control joints form a weakened plane to direct cracking to preselected locations to relieve the stress which results from shrinkage of concrete or minor temperature changes. Sawed control joints will be cut to



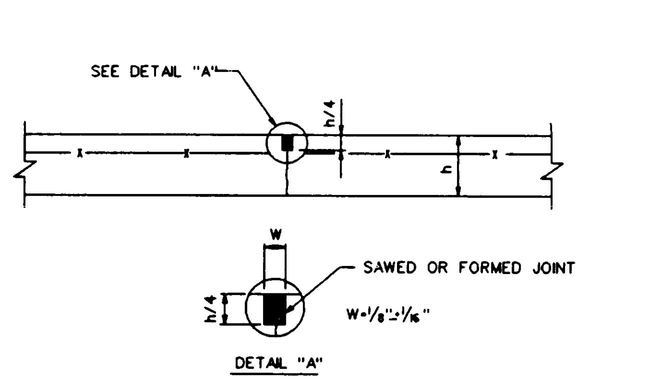
NOTES:

1. KEYED JOINT OR JOINT WITH DOWELS SHOULD ALIGN WITH AND FUNCTION AS CONTROL JOINT OR EXPANSION JOINT. USE 3/4 INCH DIA. BAR x 16 INCH LENGTH. LUBRICATE THE HALF LENGTH TO BE EMBEDDED IN ONE SIDE OF SLAB. FOR 4 AND 5 INCH SLABS, USE 16 INCH SPACING. FOR 6" SLAB, USE 12" SPACING.
2. BUTT JOINTS WITH DEFORMED TIE BARS WILL BE USED WHEN CONSTRUCTI JOINT IS NOT AT A PLANNED CONTROL OR EXPANSION JOINT LOCATION OF WHERE MOVEMENT MUST BE RESTRAINED. USE NO. 4 DEFORMED TIE BARS 30 INCH LENGTH AT 30 INCH SPACING.
3. CONCRETE COVER OF 1 1/2 INCH OR h/3 (SEE APP. B) WILL BE PROVIDED OVER REINFORCEMENT.
4. WIRE MESH REINFORCEMENT WILL BE CONTINUOUS OR INTERRUPTED WITHIN 2 INCHES OF EACH SIDE OF SLAB JOINTS AS REQUIRED FOR TYPE OF JOI BEING USED.

Figure 2-2. Construction Joints.

one-fourth depth of slab thickness. Details of control joints are shown in figure 2-3. Control joints may be made in floors scheduled to receive a floor covering by inserting fiberboard strips in the unset concrete. Depth of fiberstrip should be one-fourth of the slab thickness. Location and details of control joints will be shown on drawings.

i. Isolation joints. Isolation joints form a separation of other elements from the slab on grade and permit both horizontal and vertical relative movement. Isolation joints should be provided between the abutting faces of floor slab and fixed parts of the structure such as columns, walls, and machinery bases. At locations where slabs abut vertical surfaces, such as at interior and exterior foundation walls and column pedestals, isolation joints will ordinarily be a strip of 30-pound felt serving as a bond breaker. At exterior walls, perimeter insulation extended to the top of slab will serve the purpose. Where slabs will expand due to radiant heating systems, where slabs will be subject to extreme temperature changes, and where isolation from vibrations of machinery and equipment foundations is required, joint filler 3/8 inch or more thick will be required. Location and details of isolation joints will be shown on drawings. A typical isolation joint is shown in figure 2-4.



NOTES:

1. CONCRETE COVER OF 1 1/2 INCH OR h/3 (SEE APP. B) WILL BE PROVIDED OVER REINFORCEMENT.
2. ONE HALF OF WIRE MESH REINFORCEMENT (ALTERNATE WIRES) WILL BE INTERRUPTED WITHIN 2 INCHES OF EACH SIDE OF SLAB CONTROL JOINTS.

Figure 2-3. Control Joints.

j. Embedment of conduit and pipes. Horizontal runs of conduit and pipes will not be embedded in slabs on grade unless additional transverse reinforcement, or reinforcement and thickening, is provided over the pipe or conduit run. Embedded pipes and conduits will not cross slab joints where detrimental movement is anticipated. Where embedment is permitted, specific requirements will be indicated on the drawings. Aluminum conduit and pipes will not be embedded in any concrete structure.

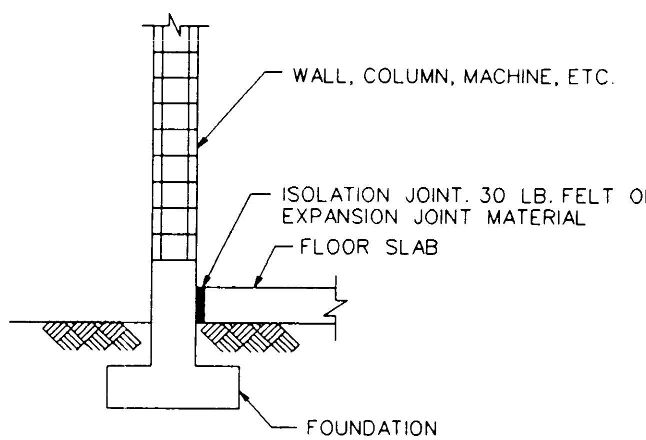


Figure 2-4. Isolation Joints.

2-13. Foundations for machinery

Machinery and generator foundations will be reinforced as required by design loads but in no case with less than 0.1 percent reinforcing each way distributed at top and bottom. Minimum bar size will be No. four, and maximum spacing of bars will be 12 inches. These foundations will be completely isolated from floor slab on grade with isolation joints. When the depth of foundation is 36 inches or more and its length-to-width ratio is 3 or more, the following criteria will also apply:

a. Longitudinal reinforcing will be distributed at top, bottom, and faces of foundation within 6 inches of the surface.

b. Horizontal bars, bent and lapped to be continuous with sidewall, top and bottom bars will be provided on end and sidewall faces. See figure 2-5 for distribution of reinforcing in heavy machinery foundations.

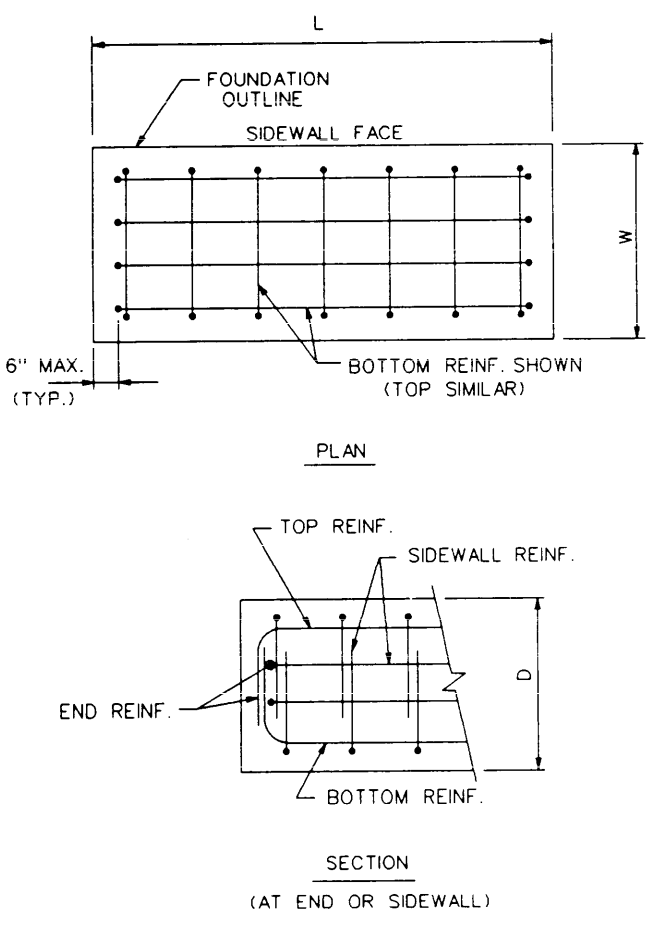


Figure 2-5. Machinery Foundation Reinforcing.

2-14. Blast-resistant construction

Design of structures to resist the effects of accidental explosions will be in accordance with TM 5-1300/AFM 88-22. The design of blast-resistant structures must consider the transient loadings and dynamic response of the structure that result from the specified design event. Blast-resistant design is often required in conjunction with the construction of weapons system facilities, both developmental and operational, as well as for hardened structures designed to resist the effects of intentional attack.

2-15. Concrete protection for reinforcement

a. Normal exposure conditions. Related criteria cited in ACI 318 will apply.

b. Seawater exposure. Seawater exposure requires that certain measures be taken to improve the protection of reinforcement. These measures can include increased cover, use of Type II or Type V cement, protective coatings, and other responses depending on the severity of exposure. See NAVFAC DM-25.06 and ACI SP-65 for additional discussion and guidance concerning concrete exposed to seawater.

c. Severe exposure conditions. Where the following conditions exist, consider using increased cover or modifying the concrete mix to decrease permeability as protection against corrosion of reinforcement. Also, check provisions of design standards relating to crack control and, if use of smaller bars or lower stresses is impractical, consider using surface coatings or impregnations to decrease the permeability of the concrete or using coated (zinc or fusion bonded epoxy) reinforcement. (A series of pullout tests conducted by the National Bureau of Standards has indicated that bars with up to 10 mils of epoxy have essentially the same bond strength as uncoated bars. Bars coated with 25 mils of PVC, however, showed considerably reduced bond strength.) Severe exposure conditions include, but are not limited to the following:

(1) Tropical climate coupled with exposure to offocean wind. This condition applies primarily for exterior exposures, although interior exposures will be included if interior spaces are not environmentally controlled. (Recommended practice is 2 inches clear cover over main reinforcement for tops of roof decks, exterior walls above grade, and for slabs on grade; 2-1/2 inches for exterior walls below grade; and 1-1/2 inches for other elements.)

(2) Industrial atmosphere.

(3) Locomotive blast.

(4) Chemical attack (including alkali).

2-16. Detailing requirements

Details and detailing of concrete reinforcement will conform to ACI 315. Engineering and placing drawings for reinforced concrete structures will conform to ACI 315R. For seismic areas, the design and details will conform to TM 5-809-10/AFM 88-3, Chapter 13. Special attention will be given to detailing reinforcing in brackets or column heads which provide bearing for simply supported members to preclude splitting or breakoff of the bearing surface. Construction and crack control joint locations will be shown on the drawings.

CHAPTER 3

STEEL

3-1. Introduction

This chapter prescribes criteria for the design of structural steel, open-web steel joists, and cold- formed steel structural members for buildings.

3-2. Basis for design

a. General. Structural framing systems and elements of buildings will be designed in accordance with the accepted industry standards listed below. The type of steel and unit dimension, (bay size, story, height, etc.), the system for structural framing, and the design method used will be based on a comparative economic study and will be those that result in the least cost for the required structure.

b. Buildings.

(1) Structural steel.

(a) Allowable stress design (ASD). The American Institute of Steel Construction (AISC) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings by allowable stress methods (including supplements), the AISC Code of Standard Practice, and referenced standards published by the AISC will apply.

(b) Load and resistance factor design (LRFD). The new AISC LRFD manual is based on the philosophy of using separate factors for each load type and for each resistance mechanism rather than the generally used allowable stress design (ASD) method. The ASD method is characterized by the use of unfactored "working" loads in conjunction with a single factor of safety applied to resistance. The new principle provides a more uniform reliability for all steel structures under various loading conditions. Since most applicable building codes have recognized or will recognize LRFD as an alternate steel building design method to the current allowable stress design rules, use of LRFD will be allowed as an optional design method for military design projects.

(c) Plastic design. Analysis and design of steel structures on the basis of plastic design, as described in ASCE Plastic Design in Steel and AISC Plastic Design of Braced Multistory Steel Frames may be followed.

(2) Open-web steel joists and joist girders. Open-web steel joists and joist girders will be designed in accordance with Steel Joist Institute (SJI) Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders.

(3) Cold-formed steel structural members. Cold-formed steel structural members to be used in buildings will be designed in accordance with the American Iron and Steel Institute (AISI) Specification for the Design of Cold-Formed Steel Structural Members.

(4) Steel deck for floors and roofs. Steel deck for floors and roofs will be designed in accordance with the Steel Deck Institute (SDI) Design Manual for Composite Decks, Form Decks and Roof Decks.

(5) Steel cables. Steel cables will be designed in accordance with the AISI Manual for Structural Applications of Steel Cables for Buildings. The factor of safety will be increased up to 5 (based on ultimate) for threaded hardware where wind, earthquake, or other vibratory loadings may occur.

(6) Metal building systems. Refer to chapter 7 for design criteria for metal building systems.

(7) Crane runways and supports.

(a) Stops and bumpers. As used in the following discussion, stops refer to rigid assemblies installed at the ends of crane runways to prevent traveling cranes from running beyond the ends of the runway. Bumpers refer to those devices (usually fitted onto the crane) which are of resilient or other energy absorbing construction designed to limit the deceleration force resulting from the crane's hitting the runway stops. Removable stops or bumpers will be provided at ends of runways. If the stop engages the tread of the wheel, the height will not be less than the radius of the wheel. Stops engaging other parts of the crane are recommended. Requirements for the design of crane stops are controlled largely by the design of the crane bumpers. Toward that end, procurement documents for cranes will mandate that crane bumpers be designed in accordance with requirements of the Occupational Safety and Health Act (OSHA) including:

- Bumpers will be capable of stopping the crane (not including lifted load) at an average deceleration of no more than 3 feet per second per second with the crane traveling at 20 percent of rated speed.
- Bumpers will, at a minimum, have sufficient energy absorbing capacity to stop the crane when it is traveling at 40 percent of rated speed. The forces to be resisted by the stops will either be indicated by the crane manufacturer or determined as set forth in Whiting Corporation Overhead Crane Handbook.

(b) Deflections. Vertical deflection of crane runway girders will be limited as set forth in Crane Manufacturer's Association of America (CMAA) 70 and 74. Horizontal deflection will be checked to assure compatibility with clearance between flanges of double-flanged wheels and bearing area of single- flanged wheels.

3-3. Design for corrosive conditions

a. General. When steel members are to be exposed in areas of heavy industrial pollution, salt spray, salt air, or are to be embedded in corrosive soils, a corrosion engineer will be consulted to recommend materials, protection, or both and to review design drawings to assure the durability of the structure. When appropriate, an increased thickness, i.e., corrosion allowance, will be used to attain the required service life. For additional discussion and guidance in designing for corrosive conditions, see the National Association of Corrosion Engineers (NACE) Corrosion Handbook and other relevant NACE publications as well as NAVFAC M0-306.

b. Design provisions. Conform to the following where applicable:

(1) Box-shaped members will be designed so that all inside surfaces may be readily inspected, cleaned, and painted, or they will be closed entirely.

(2) In outdoor structures, the flanges of two (back to back) angle members, if not in contact, will have a minimum separation of 3/8 inch.

(3) Pockets or depressions in horizontal members of outdoor structures will have drain holes or will be filled with concrete, mastic, or grout. Positive drainage will be provided away from exposed steel. Column bases will be terminated on concrete curbs or piers above grade, and tops of curbs or piers will be pitched to drain.

(4) Steel sheet piling should be capped with concrete to eliminate rapid corrosion of the exposed end.

(5) Where extremely corrosive conditions exist, consideration should be given to providing cathodic protection in addition to protective coatings for steel piling exposed under water.

(6) If cathodic protection is not installed initially, consideration should be given to providing bonded cables to simplify future connection of cathodic protection for steel piling exposed under water.

(7) Structural members embedded in concrete and exterior railing, handrails, fences, guardrails, and anchor bolts will be coated with zinc or coal tar enamel. Coating will extend at least 6 inches beyond the embedded portion. Where coated anchor bolts are used, a reduction in bond strength when estimating pull out resistance should be considered.

(8) Isolation of dissimilar metals, e.g., aluminum and steel, will be by use of fiber or plastic pads, by use of nonmetallic coatings, or by other means as approved by the appropriate headquarters.

c. Use of corrosion-resistant steel conforming to ASTM A *242 or A 588 (weathering steels).*

(1) These steels will not be used in an unpainted condition in areas where the atmosphere contains salt spray. This applies even to locations remote from a body of salt water if experience demonstrates that the prevailing winds can carry salt-laden air into contact with the structure.

(2) These steels should not be used in a seawater environment. Only limited benefit, e.g., improved fatigue resistance, will be derived. Plain carbon steel (ASTM A 36) is frequently a better choice.

(3) These steels will not be used in buried structures unless coated.

d. Use of steel conforming to ASTM A 690. See NAVFAC DM-25 for requirements regarding use of this type steel.

e. Use of stainless steel (ASTM A 666).

(1) These steels will not be used in salt spray zones.

(2) These steels will not be used if buried or if likely to become buried. In general, accelerated corrosion will occur under washers in an aqueous environment if contact with oxygen is precluded.

(3) See AISI Stainless Steel Cold-Formed Structural Design Manual as the basic design reference for stainless steel construction.

3-4. Expansion and contraction joints

Expansion and contraction jointing of steel structures, including details around embedded columns and clearance over and adjacent to nonbearing construction, will be such that neither temperature nor lateral or vertical deflection movements of the structural system will adversely affect the related construction. A double column arrangement is the preferred method of establishing an expansion joint. Consideration will be given to the effects of lateral movement of long structures due to thermal expansion and contraction. Expansion joints for buildings will be provided as set forth in chapter 2. However, spacing may be increased based on any special conditions of the local climate, exposure, or type of framing system selected. Additional expansion joints will be provided at the junctures of T-, L-, U-shaped and other irregularly shaped buildings and when a change in the type of foundation construction creates potential for significant differential settlement which could impact the building framing system.

3-5. Requirements for wear protection

The total thickness of design sections subject to wear will be increased beyond that required to meet stress requirements. The amount of such increase will be based on the material involved, the frequency of use, and the designed service life. Estimates of the wear requirement will be based on previous experience or accepted practice for the application. Use of replaceable wear plates should be considered where extremely severe conditions exist.

3-6. Climatic considerations

a. Arctic and Antarctic zones. For carbon steel, the transition from ductile to brittle behavior occurs within temperatures to be expected in Arctic and Antarctic zones. The lower ductility normally is not important, but if toughness is critical as for bridges, nonredundant, or other special structures, consideration will be given to the requirements of ASTM A 709. In addition, and especially if obtaining proper steel to minimize toughness concerns is unlikely, structures subject to impact should be designed considering the following factors:

(1) Avoiding "stress risers" is important. As examples:

(a) Provide ample fillets.

(b) Two thin plates with welds offset are better than one thicker plate with a thicker weld.

(2) Welding is a particular concern. Use bolted joints if feasible. Take precautions to eliminate gas and impurities in welds. Proper preheating and post cooling are essential.

(3) Low-carbon steels and nickel-alloy steel show better toughness at low temperature than do carbon steels.

b. Tropical zones. The effects of increased temperatures, as in tropical zones, on the load capacity of steel members may be neglected.

c. Steel exposure to weather. Structural steel for buildings will be permanently exposed to weather only where overall efficiency and economy result and where the maintenance cost of the exposed steel has been fully evaluated and is acceptable or where exposed steel is required by the architectural concept. Weathering steel may be used subject to providing means to control or conceal the effects of runoff staining of surrounding surfaces which occurs during early exposure of weathering steel. Temperature effects due to exposure of the structural steel framing system will be analyzed and necessary adjustments included.

3-7. Elevated temperatures

a. Hot-rolled carbon steel. Up to 150 degrees F, strength of steel will be assumed to be the same as the strength at normal temperature. Above 150 degrees F, the yield strength decreases with increasing temperature. See table 3-1 for properties of steel at elevated temperatures. For steels not listed, a manufacturer will be consulted to obtain strength values for elevated temperatures.

b. High strength and heat treated steels. The effect of elevated temperatures on high strength and heat treated steels should be thoroughly investigated. For example, quenched and tempered materials will undergo radical changes in their mechanical properties as well as toughness when subjected to temperatures above 500 degrees F.

Table 3-1. Properties of steel at elevated temperatures.

Type of Steel	Temperature in Degrees	Yield Strength 0.2% Offset (ksi)	Tensile Strength (ksi)	Elongation (in 2 in.) (%)
ASTM A 36	80	36.0	64.0	37
	300	30.2	64.0	25
	500	27.8	63.8	28
	700	25.4	57.0	35
	900	21.5	44.0	42
	1100	16.3	25.2	50
	1300	7.7	9.0	71
ASTM A 242	80	54.1	81.3	31
	200	50.8	76.2	31
	400	47.6	76.2	27
	600	41.1	81.3	24
	800	39.9	76.4	28
	1000	35.2	52.8	21
	1200	20.6	27.6	48
				Elongation (in 1 in.) (%)
ASTM A 588 (High Temp)	80	58.6	78.5	30
	200	57.3	79.5	28
	400	50.4	74.8	24
	600	42.5	77.7	24
	800	37.6	70.7	25
	1000	32.6	46.4	26
	1200	17.9	23.3	48

3-8. Connections

a. *Bolted connections*. There will be a minimum of two fasteners in any connection (not including pinned or welded connections) except for secondary bracing members such as lacing and battens and except for incidental connections (not including primary bracing members) not proportioned on the basis of calculated stress.

b. Welded connections. Welded connections will comply with the AISC Manual of Steel Construction and American Welding Society (AWS) D1.1.

3-9. Structural steel identification

a. Nomenclature. In response to a need to standardize the designations and nomenclature of structural steel shapes, AISI developed a system of designations. In general, the designations employ an initial letter to indicate the shape followed by numerals to indicate the size and weight. Typical examples are:

Designation
W27X114
S24x90 C10x30
L4x3x1/4
WT18x80

Proper designations for structural steel shapes are listed in the AISC Manual of Steel Construction.

b. Identification. ASTM A 6 defines the following color system to identify structural steels :

A 242	Blue
A 283 Grade D	Orange
A 514	Red
A 529	Black
A572 Grade 42	Green & White
A 572 Grade 50	Green & Yellow
A 572 Grade 60	Green & Gray
A 572 Grade 65	Green & Blue
A 588	Blue & Yellow
A 709 Grade 50	Green & Yellow
A 709 Grade 50 W	Blue & Yellow
A 709 Grade 100	Red
A 709 Grade 100W	Red & Orange

ASTM A 500, A 501, and A 618 structural steel tubing are marked by rolling, die stamping, ink printing, or paint stenciling to show the manufacturer's name, brand, or trademark; size and thickness; specification number; and grade letter.

3-10. Cross reference to steels

A guide for a cross-reference to steels is given in American Society for Metals (ASM) Engineering Properties of Steel.

CHAPTER 4
WOOD

4-1. Introduction

This chapter prescribes criteria for the design of wood buildings. Properties of wood and other considerations influencing design including design of plywood elements and built-up members, wood preservation, termite control, fire retardant treatment, and climatic influences are presented. References are made to appropriate design standards and specifications. The use of timber construction will consider type of occupancy and will meet the fire protection criteria and requirements set forth in MIL-HDBK 1008A.

4-2. Basis for design

a. Lumber and timber. Lumber and timber include gluedlaminated members. The design of structural lumber and gluedlaminated timber, except as modified in this manual, will be based on the American Institute of Timber Construction (AITC) Timber Construction Manual. The National Forest Products Association National Design Specification for Wood Construction is by reference a part of the AITC Timber Construction Manual and gives detailed criteria for designing engineered timber construction. Redwood will be used structurally only in cooling towers.

(1) Sealers. Where feasible, seasoning checks in the ends of timber pieces installed in an unseasoned condition should be minimized by the use of end coating or sealers.

(2) Connections. Connections will be detailed to permit periodic tightening.

(3) Hardware. Bolt holes for drift bolts will be bored with a bit having a diameter 1/8 inch less than the bolt diameter.

b. Plywood. Plywood elements will be designed according to the American Plywood Association (APA) Plywood Design Specification. Fire-retardant treated plywood will not be used in Army facilities except in nonstructural applications which are not subject to elevated temperatures or high humidity. Fire-retardant treated plywood will not be used in any part of the roof or roofing system.

c. Lightwood trusses. Light metal plate connected wood trusses or glued-nailed trussed rafters may be used as lightwood trusses in roof construction.

(1) Light metal plate connected wood trusses will be designed so that they conform to Truss Plate Institute (TPI) Design Specification for Metal Plate Connected Wood Trusses. Care will be taken to assure that trusses are fabricated in conformance with TPI Quality Standard for Metal Plate Connected Wood Trusses.

(2) Glued-trussed rafters will be designed according to Midwest Plan Service, NWPS-9, or the method and data described in the Purdue University Agricultural Experiment Station Research Bulletins No. 714 and No. 727.

(3) Wood trusses will be braced in accordance with TPI Commentary and Recommendations for Bracing Wood Trusses.

(4) Provisions will be made to assure that handling and erection of wood trusses is performed according to TPI Commentary and Recommendations for Handling and Erecting Wood Trusses.

d. Bridging. Floor joists, roof joists, and rafters of buildings will be laterally supported by permanent cross bridging installed at intervals not exceeding 8 feet. Under the following conditions, bridging may be omitted for design floor loadings of 40 pounds per square foot or less and clear spans of 15 feet or less:

(1) When tongue and groove wood strip finish flooring is installed at right angles to joists over a subfloor.

(2) When 25/32-inch tongue and groove wood strip finish flooring is installed at right angles directly to joists spaced at 16 inches on centers.

(3) When another type of finish floor is installed over a plywood underlayment which is applied over a subfloor.

4-3. Design stresses

a. Structural and framing members. Design allowable stresses for wood used for structural and framing members will be in accordance with National Forest Products Association National Design Specification for Wood Construction and Wood Structural Design Data. Design allowable stresses will be established from the National Forest Products Association documents based on species and grade of the material being used. Listed below are criteria to establish the minimum acceptable quality for materials to be used in the applications indicated.

(1) For single-member uses, the minimum allowable unit stresses for stress-grade lumber will be 1,000 pounds per square inch (psi) for Fb, extreme fiber in bending; 525 psi for Ft, tension parallel to grain; 875 psi for Fc, compression parallel to grain; and 1,200,000 psi for E, modulus of elasticity.

(2) For repetitive-member uses, the minimum allowable unit stresses for stress-grade lumber will be 1,200 psi for Fb, extreme fiber in bending, and 1,200,000 psi for E, modulus of elasticity.

b. Plywood. Design stresses for plywood used structurally will be in accordance with the allowable unit stresses in APA Plywood Design Specification.

c. Structural glued-laminated members. Design stresses will be in accordance with the AITC Timber Construction Manual.

4-4. Selection of species and grade

The following criteria will be used to select lumber species and grade for structural members of all projects: suitability for the proposed structure, economy, and availability in the particular project location.

a. Domestic species of lumber.

(1) Stress grade lumber. Design values will be in accordance with the species and grade selected from National Forest Products Association National Design Specification for Wood Construction. Where preservative treatment is required, selection of species should consider ease and effectiveness of treatment.

(2) Nonstress grade lumber. Nonstress grade lumber may be used for miscellaneous framing such as nailers, caps, bucks, grounds, sleepers, blocking, bridging, plates, and furrings. Such members will be standard grade or better. Lumber to be used for studs will be stud-grade or better.

(3) Plywood. Plywood for general application will be of species groups 1, 2, or 3, as classified in the APA Plywood Design Specification. Plywood of species group 4 may be considered for applications such as siding where it is not designed to carry calculated stresses.

(4) Durability. See United States Department of Agriculture (USDA) Wood Handbook for durability data.

(5) Grade marking. Domestic lumber and timber should be grade marked by an agency recognized by the American Lumber Standards Committee.

b. Nondomestic species of timber.

(1) Properties. Many nondomestic species of timber are suitable for construction work. Some have very high strength and are more durable than softwoods. Resistance to attack by marine borers is claimed for some species, but performance data suggest that such resistance is not reliable.

(2) Applications. Items of particular interest concerning application of these species are:

(a) Pressure preservative treated Apitong (dipterocarpus grandiflorus bianco) is highly suitable for wood piling and utility poles.

(b) The tropical woods Ipil (Instia bijuga), Daog or Palomara (Calophyllum inophyllum), Ahgao (Premna obtusifolia), Fago (Ocbrosia oppositifolia), Yacal (Hopea, Sborea, and Isoptera species), Molave (Vitex parviflora Juss), and Chopag (Ocbrocarpos odoraturus) are satisfactory for most structural uses.

(c) The following tropical woods, on Guam, should be used only when construction is to be of a temporary nature: Coconut (Cocs nucifera), Dugdug (Artocarpus sp.), Nunu (Ficus prolixa), Yoga (Elacocarpus joga), and Faya (Tristiropsis obtusangula).

(3) Allowable stresses. Strength properties of individual nondomestic species should be obtained from the potential supplier. Allowable stresses should be one-fourth to one-third of the ultimate strengths. The designer should regard characteristics as published by the supplier with caution and should insist on tests of random specimens to verify assumed strength characteristics.

4-5. Climatic considerations.

Climatic influences for cold and tropical regions are as follows:

a. Cold region conditions. For cold region limitations, see TM 5-852-9/AFR 88-19, Volume IX for Army or Air Force projects. Engineering properties usually are not appreciably affected when wood is subjected to extremely low temperatures.

b. Tropical Conditions. Engineering properties of wood are not appreciably affected in tropical climates. Rot and insect attacks, however, are aggravated in tropical humid areas, and all timber for permanent construction in tropical areas should be preservative treated except local native hardwood as discussed herein. Structural bonding to other materials should be by means of epoxy resin adhesive. Bonding of wood to wood can be made by a variety of adhesives, such as those covered in Mil. Spec. MIL-A-22397 for marine or severe outdoor use and Fed. Spec. MMM-A-181 for general purposes.

c. Abnormal wetting and drying of floors. In areas of buildings where floors are subjected to abnormal wetting and drying, wood framing members will be set on foundation walls or piers at least 4 inches above the floor level.

4-6. Fire retardant treatment

Recommendations in USDA Wood Handbook and in National Fire Protection Handbook should be followed. Pressure impregnation is the preferred treatment. Also, see National Fire Protection Association 703.

4-7. Termite control

Termite control measures will be used in areas prone to termite infestation. Soil will be treated with commonly accepted products prior to construction.

4-8. Special consideration

a. Anchorage. Roof systems will be adequately anchored to their support to prevent uplift and shear from wind and seismic forces. Exterior frame walls will be securely attached to the foundations. Minimum anchorage will be 1/2-inch diameter steel bolts not more than 4 feet on center. Although 1/2-inch bolts are sufficient , anchor bolts less than 3/4-inch diameter are not recommended, due to the possibility of stripping the threads of the bolt with hand wrench and the difficulty of replacing anchor bolts embedded in concrete.

b. Bolt replacement. Bolts will be embedded in cast-in-place concrete to the minimum depths indicated in table 2-2 and not less than 15 inches in unit masonry walls and piers. Bolts will be placed not more than 12 inches from corners of the building and not more than 12 inches from the ends of sill members. Each bolt will be provided with at least one standard 2-inch diameter washer or a 2-inch square washer, 1/8 inch thick, or the equivalent.

4-9. Built-up members

Built-up members include those which are mechanically connected assemblies of smaller timber elements designed to act as a single unit. The effectiveness of a mechanically connected, built-up member is, in part, related to the orientation of the interfaces between its component parts and the direction of load application. It is generally accepted that mechanically connected, built-up members are not as effective against load, deflection, or both as are solid timbers or glued, built-up members. This type construction is seldom used in current construction and will be considered only when more conventional approaches are not available. When used, machanically connected, built-up members will be designed in accordance with National Forest Products Association National Design Specification for Wood Construction and accepted industry practice. Particular attention will be given to the details of interconnecting components parts so that the assembly will act reliably as a unit.

4-10. Plywood members

a. Built-up plywood girders. The following precautions will be taken in designing built-up plywood girders:

(1) Allowable shear stress between flanges and web will not exceed 0.14 times allowable stress in horizontal shear.

(2) Web stiffeners will be screwed or glued to webs and in contact with both flanges. The stiffener thickness will be at least 6 times the thickness of the web; the width/thickness (b/t) ratio of the stiffener will not exceed 8; and the minimum stifferner thickness will be 3/8 inch. Stiffeners will be as wide as the flange. Spacing will be equal to or less than 2 tYmes the clear distance between flanges.

(3) Wood blocks (bearing stiffeners) will be provided at points of concentrated load or bearing, or both.

(4) For deep girders, allowable stresses will be reduced to account for lack of lateral support of the center fibers as compared to the flange fibers.

(5) Web material should have species group I face plies but need not necessarily conform to marine and stuctural I grades.

b. Stressed-skin panels. In bending , tension, and compression, consider only those plies where the grain is parallel to the span.

c. Exposure. Where exposed to weather or in humid location, e.g., toilets and shower rooms, use exterior grades of plywood.

4-11. Glued, built-up (including laminated) members

Design standards, procedures, and provisions for individual components (whether plywood or sawed lumber) will conform to the requirements for such components as previously indicated, except as follows:

a. Transverse joints. Transverse joints in the planks may be considered as transmitting stress if finger joints having a slope no steeper than 1 to 10 are used. Joints will be spaced not less than 24 times the lamination thickness in areas of maximum stress. In lesser stressed areas, spacing may be reduced linearly in proportion to relative stress. Butt joints will not be used for structural members.

b. Mechanical fasteners. Mechanical fasteners may be used in fabricating glued construction as necessary to hold connected elements until the adhesive cures. Mechanical fasteners , however , will not be considered as transferring any portion of the required stress when present in conjunction with adhesive. The movements required to develope the stength of mechanical fasteners are inconsistent with those permitted in giued joints

c. Exposure. Glued-laminated members may be used in exterior exposure and under conditions of exposure to moisture and biologically destructive agents. When used in such applications, however, the adhesives and perservative treatment used will be appropriate for the conditions of exposure. See AITC Timber Construction Manual.

4-12. Treated wood members

The use of treated timbers is recommended under the following conditions subject to the following requirements.

a. Preservative treatment. Preservative treatment will be in accordance with the American Wood Preservers Association (AWPA) Book of Standards. Note the following recommendations.

(1) Cerosote and creosote soulutions are not recommended where color, odor, or exudation of the preservative may be undesirable. Waterborne or oilborne preservatives in volatile solvents should be used.

(2) Where cleanliness and paintability are required, preservatives should be of the waterborne type or the oilborne type in volatile solvents.

(3) Quality marking should be provided on all treated lumber.

b. Structural framing. Pressure preservative tretment for timber should be used under the following conditions of exposure:

(1) All wood in contact with ground or water.

(2) Wood in contact with masonry or metal, where conduction or condensation creates problems.

(3) Roof structures (framing and sheathing) installed over enclosed swimming pools or in building structures where high humidities prevail.

(4) Areas in or near shower rooms, galleys, sculleries, lundry rooms, and cold-storage rooms.

(5) Areas of basementless buildings in close proximity to the soil, where moisture and termites can attack structural elements.

(6) All lumber within 18 inches of the ground in slab-on-ground or crawlspace houses (basementless).

(7) All structural wood members in regions where dry-wood termites prevail.

(8) On waterfront structures, as specified in NAVFAC DM-25.

c. Fabricate before treatment. To the extent practicable, framing for treated wood structures should be prefabricated before treatment. For assemblages, consider the feasibility of fabricating and assembling the structure, then disassembling, treating, and reassembling. When unavoidable, field cuts, field-holes, etc., will be field treated with preservative before assembly.

d. Site requirements. Staff biologists should be consulted during the planning, design, and construction stages for information on wood-destroying pests at the specific site.

4-13. Drawings

Structural drawings will include design loads, design stresses, camber, and other information needed for fabrication.

4-14. Typical details

"Typical Construction Details" as shown in the AITC Timber Construction Manual, will be the basis for detailing. Details unique to each structural systems will be developed separately.

CHAPTER 5

ALUMINUM

5-1. Introduction

This chapter prescribes the criteria for designing aluminum structural systems and members for buildings. The contents cover general topics related to aluminum structural design for buildings such as connections, bending stresses, crack control, and corrosion problems. A discussion of specific cautions and special design considerations is also included.

5-2. Basis for design

Structural framing systems and elements of buildings will be designed in accordance with the following accepted Aluminum Association standards: Specifications for Aluminum Structures, Commentary on Specifications for Aluminum Structures, Engineering Data for Aluminum Structures, Aluminum Standards and Data, and Illustrative Examples of Design. Aluminum Association Specifications for Aluminum Structures using allowable stresses for building type structures will be used.

5-3. Design considerations

a. Modulus of elasticity. The modulus of elasticity of aluminum is about 1/3 that of steel, therefore aluminum requires special investigation of deflection, local and overall crippling, and buckling situations. Ductility is about three times that of steel. Refer to the Aluminum Association Specifications for Aluminum Structures for consideration of buckling and crippling modes of failure.

b. Yield strength. There are varying yield strengths for aluminum alloys. Yield strengths for aluminum alloys are based on 0.2 percent offset since the stress-strain curve has no well defined yield point like mild steel. Since there is often (depending on alloy) very little difference between yield and ultimate strength of aluminum, designs should particularly consider secondary stresses. Such stresses have cumulative effects with each other and with primary stresses without the potential for relief normally associated with yield of materials. In addition, stress risers such as notches should be avoided.

c. Welding heat. Welding heat lowers the strength of most aluminum alloys in a region within about 1 inch of the weld. All aluminum alloys are not weldable. Weldability of the material being considered will be verified.

d. Coefficient of thermal expansion. The coefficient of thermal expansion of aluminum is about twice that of steel. Because of a lower modulus of elasticity, however, stresses in aluminum alloy structures resulting from temperature changes or misalignments of parts often are lower than those in steel structures.

e. Compatibility. Composite action or interaction with steel or concrete framing involves problems of incompatibility due to the difference in coefficients of thermal expansion, modulus of elasticity, and chemical reactivity.

f. Fire protection. Published fire resistance ratings of aluminum structural elements are unavailable. Except in special circumstances, aluminum should not be used in primary structural elements where fire resistance ratings are required since some alloys start to lose strength at temperatures as low as 200 degrees F.

5-4. Expansion and contraction joints

Criteria for expansion and contraction joints will be similar to those requirements for steel in chapter 3.

5-5. Selection of alloy

a. General characteristics. General characteristics of all aluminum alloys include the following:

(1) Light weight.
(2) Ease of workability, fabrication, and extrusion.
(3) Corrosion resistance.
(4) Low maintenance cost.
(5) Lack of spark generation (with most materials).
(6) High electrical and heat conductivity.
(7) High reflectivity of light in visible and infrared wavelengths.

b. Structural alloys. Alloys used for structural applications of aluminum will be limited to those for which design specifications are available. For comparative characteristics of these alloys, see Table 1 of Aluminum Association Engineering Data for Aluminum Structures.

5-6. Design for corrosive conditions

a. General. Aluminum has a higher resistance to corrosion than the usual alloys of structural steel, but it is not an assurance against corrosive attack. For example, aluminum embedded in or in contact with concrete, other alkaline materials, or dissimilar metals (such as steel) under moist conditions will corrode unless isolation is provided. See the Aluminum Association Specifications for Aluminum Structures for precautions and requirements. It should also be noted that if oxygen is precluded from contact with aluminum (buried or under plastic washers) accelerated corrosion can occur.

b. Marine environments. A marine environment can be particularly detrimental to some alloys. Use caution and consult Inco-Limited Useful Guidelines in the Selection of Corrosion Resistant Materials for Marine Service, Marine Technological Transcript 3. Certain alloys (principally in the 5000 series, e.g., 5086, 5456, and 6061) have given excellent service when of proper temper.

c. Protective treatment. Protective treatments such as cladding, anodizing, special coatings, and others will be considered for treating aluminum exposed to corrosive conditions. Refer to publications of the Aluminum Association and NACE for further discussion and guidance.

d. Isolation. Aluminum will be isolated or otherwise protected in applications involving contact with dissimilar metals, with concrete, or with alkaline materials such as soap, detergents, ashes, or lye. Isolation from dissimilar metals will be achieved by coating the contact surfaces with a heavy coating of zinc chromate pigment, synthetic resin type paint. Aluminum surfaces which are to be in contact with concrete or mortar will be given a heavy coat of alkali resistant bituminous paint. Refer to Aluminum Association Specifications for Aluminum Structures for specific guidance in safeguarding aluminum in contact with dissimilar materials.

5-7. Connections

a. Minimum connections. There will be a minimum of two fasteners in any connection (not including pinned or welded connections) except for secondary bracing members such as lacing and battens and except for incidental connections (not including primary bracing members) not proportioned on the basis of calculated stress.

b. Steel bolts. Stainless steel bolts (and washers) may be used in aluminum structures without precautions for corrosion isolation. Steel bolts (not stainless) may be used if galvanized or cadmium plated and if fitted with galvanized or cadmium-plated washers. Cadmium plating is not advised for exterior exposures unless the cadmium coating is applied by painting.

c. Eccentricity. Effects of eccentricity in connections will be considered as set forth in the Aluminum Association Specifications for Aluminum Structures.

d. Installation. Care should be taken not to over torque aluminum bolts. Molybdenum disulfide may be used as a lubricant for threads to minimize the required torque.

e. Welding. Welding design and details of welding will be in accordance with AWS D1.2.

CHAPTER 6

METAL ROOFING AND SIDING

6-1. Introduction

This chapter prescribes the criteria and procedures for the design of metal roofing and siding for buildings.

6-2. Definitions

Metal roofing and siding are cold-formed, fluted, metal sheets designed to support loads perpendicular to the plane of the sheets. Normally the sheets are used in all metal structures and are installed exposed to the weather. Roofing as defined in this chapter will not be construed as sheet metal roofing to be placed on sheathings. The various types of configurations vary with each manufacturer.

a. Regularly curved sheets. Regularly curved sheets are sheets that have conventional, smooth, regular flutes symmetrical about the neutral axis regardless of depth or curved shape. Shapes may be parabolic, sinusoidal, or any other arc- tangent type continuous curve.

b. Regularly folded sheets. Regularly folded sheets are sheets that have flutes with right-angle or slanted webs folded with or without flats symmetrical about the mid-depth axis regardless of depth or shape. Flutes may be rectangular, trapezoidal such as the ribbed type, triangular such as the V-beam type, or any other equally folded continuous configuration.

c. Irregularly folded sheets. Irregularly folded sheets are sheets with right-angled or slanted webs folded with different flat widths at the top and bottom or with flats only at the top or bottom. Depths will be uniform on major flutes with intermediate stiffened flutes permitted. The basic complete pitch of the system of flutes will be repeated regularly.

d. Standing seam metal roof system. A standing seam metal roof (SSMR) system is a unit composed of metal roof panels, attachment clips, insulation, metal deck (where used), subpurlins (where used) and fasteners that attach clips to the supporting structural members. Also included as part of the SSMR system are the ridge, gable trim, eave trim, gutters, scuppers, facia, soffit and flashing necessary to produce a complete roof system. The SSMR system elevates panel side laps above the general plane of the surrounding roof area and above the water path. Hold-down clips are frequently used to elevate roof sheets above their supporting purlins to allow insulation to pass between these two elements with minimal compression. The exterior panels of SSMR systems are not to be considered for diaphragm action.

e. Architectural standing seam roof. An architectural standing seam roof is an architectural metal roofing applied over a solid substrate underlayment. It is specified for mansards, fascias and roof slopes of 3:12 or greater. The design requirements in this manual do not apply to architectural metal roofing.

6-3. Basis for design

Unless otherwise prescribed herein, the design of roofing and siding will be in accordance with the following:

a. Steel. AISI Specification for the Design of Cold-Formed Steel Structural Members.

b. Aluminum. The Aluminum Association Specifications for Aluminum Structures.

6-4. Design stresses

The minimum yield stresses and design moduli of elasticity to be used as a basis for design will be as follows:
Steel: $Fy = 33{,}000$ psi; $E = 29.5 \times 10^6$ psi
Aluminum: Use Fy and E as determined from Aluminum Association Specifications for Aluminum Structures.
The above mechanical properties for steel are based on ASTM A 570, Grade 33, and ASTM A 611, Grade C. Properties for aluminum alloy vary with alloy type and heat treatment.

6-5. Design loads

Roofing and siding will be designed to support dead, live, and wind loads. Midspan deflections under maximum design load will be limited to the values stated below. Design of roofing will include consideration of the effects of the maximum allowable deflection of roofing under maximum design loads. The maximum net inward and outward loads used in the design will be indicated on the drawings.

6-6. Design requirements

The design of roofing and siding will conform to the design specifications and guidance referenced except as modified herein.

a. Deflection. Maximum deflection for metal roofing and siding under full dead and live and/or wind loads will not exceed 1/180th of the span between supports. Maximum deflections will be based on sheets continuous across two or more supports with sheets and fully free to deflect and rotate.

b. Thickness. The minimum thickness of sheets will be as follows:

	Roofing*	Siding
Aluminm	0.032 inch	0.032 inch
Steel-Plain	24 gauge	24 gauge
Painted Protected	(0.0239 inch)	(0.0239 inch)

*Except that roof panels will be 22 gage (0.03 inch) thick minimum in building areas with uplift pressures greater than 60 psf.

c. *Roof slopes.* Metal roofing will not be used on slopes less than 1-1/2 in 12 except as follows:

(1) Slopes as low as 1/4 in 12 may be used with mechanically crimped standing seam roofs except in highly corrosive environments where the minimum will be 1/2 in 12.

(2) Height of all corrugations at overlap for adjacent roof sheets or standing ribs of snap seam panels shall be not less than 2 inches for roof having a slope less than 2 in 12, 1 inch for roofs having a slope of 2 in 12 but less than 3 in 12, or 1/2 inch for roofs having a slope of 3 in 12 or greater.

d. Spacing of supports and length of sheets. Supports for fluted roofing and siding sheets will be laid out to provide equal or approximately equal multi-span conditions insofar as practicable considering maximum length of sheet at 30 feet for Army facilities, and 25 feet for Air Force facilities. Sheets in excess of 30 feet, although available, require design provisions for thermal expansion and contraction.

e. Diaphragms and shear walls. Panels thinner than 22 gage are not permitted for diaphragms or shear walls. The roof or wall system will have the ability to transmit diaphragm loads as dependent upon the strength, stiffness, panel configuration, fastening method, and condition of installation. The preferred methods for resisting lateral loads are cross-bracing, rigid frames, or wind columns. For buildings with cranes, roof and/or wall diaphragms will not be allowed. The exterior panels of standing seam roofs are not to be considered for diaphragm action.

f. Standing seam metal roof systems.

(1) In cases where the structure is not a complete metal building system, the supporting structural system for the roof will be designed by the engineer of record (EOR). For in-house work, the EOR is the Chief of Engineering for the office performing the design. When the supporting structural system is designed by the EOR , the supporting members should be designed running perpendicular to the SSMR. If the designing engineer is using the metal deck as a diaphragm, he will specify the properties and attachment of the metal deck. If the supplier of the SSMR system elects to submit their supporting structural system in place of the system designed by the EOR, the supplier must furnish a complete structural analysis of the supporting system to the EOR for approval. This analysis must show that the supporting structure is adequate to carry all loads imposed upon it and meet all the requirements of the design criteria. If a SSMR system deviates from any of the requirements given in subparagraph (3) through (9) below, the system will be tested in accordance with the Corps of Engineers' test (Appendix D).

(2) When designed by an A/E or an in-house designer, the design wind uplift pressure and dimensions of edge and corner zones will be shown on an isometric view of the roof which will be included on the contract drawings. When designed by a metal building supplier, the isometric view will be shown on the shop drawings submittal and the as-built drawings. This includes all areas of the roof; i.e., ridges, edges and corners.

(3) SSMR systems will have mechanically crimped sidelap seams at panel to panel connections. A mechanically crimped seam is one where the interlocking of the two panels is accomplished by a mechanical device forming or repositioning the metal. Any manufacturer wishing to use a system that does not have mechanically crimped sidelap seams must demonstrate by testing in accordance with appendix D that the sidelap seams will not open under application of the most severe factored wind uplift pressure. This data must be submitted to the EOR for review and approval.

(4) Hold down clips will be capable of self adjustment for the full anticipated thermal movement. The maximum temperature range for thermal movement will be stated in the project specifications. The clips will have the movement slots centered at the time of installation. The calculations to determine the maximum anticipated thermal movement will be included with the shop drawings. Carefully designed expansion joints may be necessary on long span roofs (100 feet or greater).

(5) There will be a minimum of two fasteners (usually screws) per hold down clip. As a confirming test, each fastener will be capable of resisting the tributary area load of the clip times the design wind uplift pressure applicable to that portion of the roof times a factor of safety of 2.25 per screw. Results of the test will be submitted along with the shop drawings.

(6) Single fasteners will be allowed if the supporting structural members are prepunched or predrilled. As a confirming test, the fastener will be capable of resisting the tributary area load of the clip times the design wind uplift pressure applicable to that portion of the roof times a safety factor of 3.0. Results of the test will be submitted along with the shop drawings.

(7) Fasteners (screws) will be equipped with metal backed neoprene, or similar type material, washers that will assure the proper torque of the screws by observation of the compression in the neoprene.

(8) The supporting structural member spacing will not exceed 2'- 6" on center at the corner, eaves, and edges of the roof and 5'- 0" maximum for the remainder of the mid field roof area. Edge members for vertical support are required on all perimeter walls.

(9) Cold formed supporting structural members will have a minimum thickness of 16 gage and a minimum

tensile yield strength of 50,000 psi. Other types of structural members will have a minimum thickness of 1/4 inch and a minimum tensile yield strength of 36,000 psi.

6-7. Minimum required section properties for fluted sheets

The minimum required values of section properties per foot width of fluted sheets for single-span, two-span, and three or more span conditions as applicable will be indicated on the drawings . For the usual roofing and siding applications, the section properties that must be indicated are the minimum effective moment of inertia per foot width, the minimum thickness of sheets, and minimum section modulus per foot width for both aluminum and steel sheets if an option is allowed. For presentation of necessary information and data on the drawings, see table 6-1.

a. Design considerations. The Blodgett formulas, equations 6-4 and 6-5, may be used to determine the section properties of regularly curved sheets. In some special cases, fluted roofing and siding may be required to perform additional structural functions, for example to provide diaphragm action when subject to design loads. The structural evaluations and design of fluted sheets in most special cases will be based on criteria described herein and on sound engineering principles. Design of diaphragms for earthquake areas will conform to TM 5-809-10/AFM 88-3 Chapter 13.

b. Determination of minimum required section properties. The minimum required section properties will be determined by the following criteria:

(1) Minimum required moment of inertia. For the usual case where the design load is a uniformly distributed load, the minimum required effective moment of inertia per foot width (I) based on the deflection limit under full load (span length, L, divided by 180) can be determined by the following formulas:

Single spa:: $$I = 2.34\, wL^3/E \qquad \text{(eq 6-1)}$$

Two equal spans: $$I = 0.97\, wL^3/E \qquad \text{(eq 6-2)}$$

Three or more equal spans: $$I = 1.24\, wL^3/E \qquad \text{(eq 6-3)}$$

Where I = Effective moment of inertia, in^4 per foot of width.
E = Modulus of elasticity, psi.
w = Uniformly distributed design load, pounds per linear inch per foot width.
L = Span, center to center of supports, inches.

(2) Minimum required section modulus. These values will be determined by dividing the maximum moment resulting from critical design loads by the allowable stresses. The allowable design stresses will be based on the maximum allowable stresses for the specific metal which will provide the minimum effective section modulus.

(3) Selection of sheets. For sheets to be acceptable, all requirements for minimum section properties – section modulus, moment of inertia, and thickness – must be met.

6-8. Determination of actual section properties of available fluted sheets

a. General. To assure complete compliance with design criteria prescribed in this manual and to provide economical design and an equitable section comparison, properties of various available fluted sheets will be determined in accordance with the procedures listed below.

b. Section properties. Section properties will be based on unclad sheet. The moment of inertia and section modulus of various fluted sheets may be determined by any of the methods specified below:

(1) Regularly curved sheets.

(a) Section properties may be determined in accordance with the following formulas derived by Professor H. B. Blodgett (see AISI Handbook of Steel Drainage and Highway Construction Products).

$$I = C_5 bt^3 + C_6 bd^2 \qquad \text{(eq 6-4)}$$

$$S = \frac{2I}{d+t} \qquad \text{(eq 6-5)}$$

Where I = Moment of inertia, in^4/ft. of width.
S = Section modulus, in.^{3}/ft. of width.
b = Width, (12 inches).
t = Thickness of sheet, inches.
d = Depth, inches (distance from top surface of crest of corrugation to the top surface of valley of corrugation).

Values for C_5 and C_6 may be determined from figures 6-1, 6-2, and 6-3. For most pitch-to-pitch ratios and web angles, the first term of the formula above for "I" may be neglected. Acceptable values of moment of inertia and section modulus of several types of fluted steel sheets are tabulated in an article from ASCE Civil Engineering magazine, "Sectional Properties of Corrugated Steel Sheets Determined by Formulas."

(b) Section properties may be determined in accordance with the "linear method" in the AISI Supplementary Information on the Specification For the Design of Cold-Formed Structural Steel Members.

(c) When the pitch of flutes (distance from center to center of flutes) equals 2.67 inches and the total depth (d+t) equals 7/8 inch, the section properties may be computed by the following formulas:

$$I = 0.15\, bd^2 \qquad \text{(eq 6-6)}$$

$$S = \frac{2I}{d+t} \qquad \text{(eq 6-7)}$$

Table 6-1. Design and property data for fluted roofing and siding.

Critical Uniform Load and Span Condition (pfs and inch)	Minimum Resisting Moments Required (in.-lbs/ft width)	Minimum Moment of Inertia per ft of width = I min. (inch ^{4}/ft)	Minimum Section Modulus per ft of width = S min. (inch ^{3}/ft)	Minimum Sheet Thickness (gauge or inch)
Roofing:				
Span ____inches				
____span condition				
____psf down (dead + live)	____positive	____	____	24 gauge steel**
	____negative			
____psf up (wind)	____positive			0.032 inch aluminum
	____negative			
Siding:				
Span ____inches				
____span condition				
____psf in (wind)	____positive	____	____	24 gauge steel
	____negative			
____psf out (wind)	____postive			0.032 inch aluminum
	____negative			

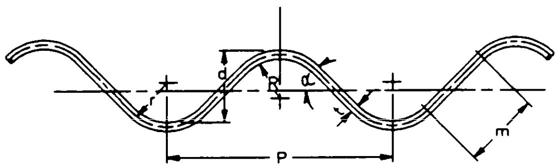
Unless otherwise indicated, the strength and deflection properties of roofing and siding will be determined by:

Steel – AISI, Specification for the Design of Cold-Formed Steel Structural Members. (Thickness used will be that of unclad sheets.) $F_y = 33{,}000$ psi* $E = 29.5 \times 10^6$ psi*

Aluminum – The Aluminum Association, Specifications for Aluminum Structures. $F_y = 27{,}000$ psi* $E = 10.1 \times 10^6$ psi*

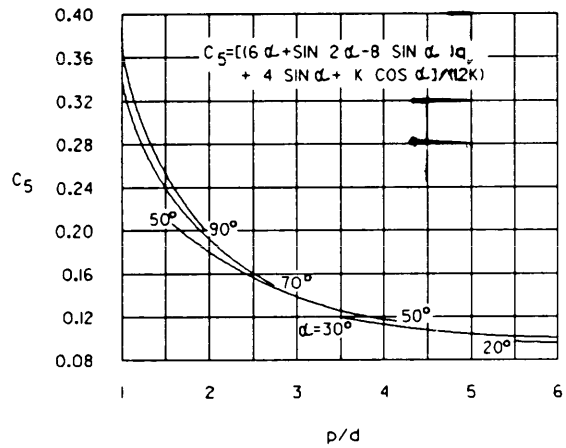
*Use values appropriate for the actual material being used.

**Minimum thickness will be 0.03 inch in the high pressure zones (corners and edges) of buildings with uplift pressures greater than 60 psf.



From: "SECTIONAL PROPERTIES OF CORRUGATED STEEL SHEETS DETERMINED BY FORMULAS" by Don S. Wolford, Civil Engineering, Feburary 1954.

Figure 6-1. Cross section of typical arc-and -tangent giving symbols used.

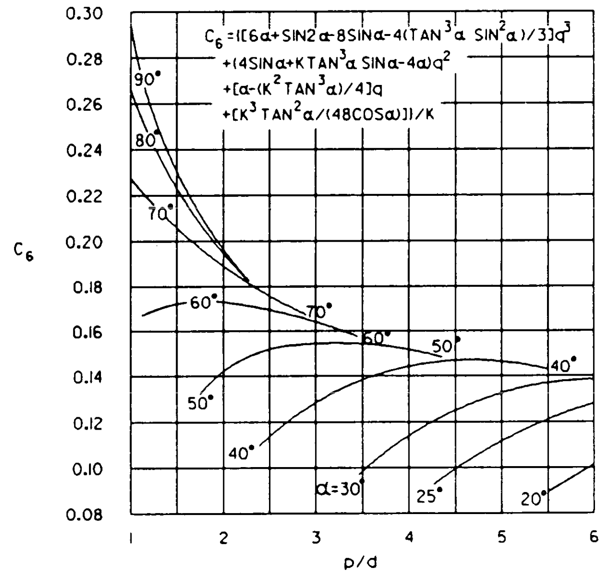


From: "SECTIONAL PROPERTIES OF CORRUGATED STEEL SHEETS DETERMINED BY FORMULAS" by Don S. Wolford, Civil Engineering, Feburary 1954

Figure 6-2. Factor C_5 plotted against pitch-depth ratio at various web angels.

(2) Regularly and irregularly folded sheets.

(a) Section properties of steel sheets will be computed in accordance with AISI Supplementary Information on the Specification for the Design of Cold-Formed Structural Steel Members or by other conventional methods based on full cross section per foot of width except where the use of the "effective design width" concept is required. The effective design width will be determined in accordance with AISI Specification for the Design of Cold-Formed Steel Structural Members. The maximum compressive stress, f, to be used for determination of the effective width under "load determination" in Section 2.3.1.1 of the AISI specification will be taken as the actual stress under the design dead plus live load but not more than 20,000 psi maximum, regardless of the actual stress and regardless of the increase of allowable stresses for wind or seismic loads. For deflection, the effective width under the "deflection determination" formula will be



From: "SECTIONAL PROPERTIES OF CORRUGATED STEEL SHEETS DETERMINED BY FORMULAS" by Don S. Wolkford, Civil Engineering, Feburary 1954.

Figure 6-3. Factor C_6 plotted pitch-depth ratio at various web angles.

used at the maximum computed compressive stress when unfastened sheet is subjected to the maximum design moment.

(b) Section proterpies of aluminum sheet will be computed by conventional methods based on full cross section per foot of width except for deflection where the effective width concept will be used as required in accordance with the Aluminum Association Specification for Aluminum Structures. Acceptable section properties of various available aluminum building products are tabulated in table 6-2.

6-9. Construction considerations to ease design

Avoid locating mechanical and electrical equipment on and penetrating through the roof. If it is absolutely necessary to locate the equipment on the roof it should be supported high enough above the deck to facilitate future repairs under the units. Walkways will be provided if access to the equipment is a requirement.

6-10. Load test

a. General. In special cases where compositions or configurations of the fluted sheets are such that values cannot be properly calculated for the safe load-carrying capacity and deflection in accordance with the procedures

Table 6-2. Nominal section properties of aluminum building products.

Product	Thickness, inch	Weight, lb. per sq ft	Area, in.^{2} per ft. of width	Moment of Inertia, in.^{4} per ft of width	Minimum Section Modulus, in.^{3} per ft of width	Maximum Section Modulus, in.^{3} per ft of width	Radius of Gyration, inch
Corrugated Roofing and	0.032	0.552	0.469	0.0409	0.0936	0.0936	0.295
Siding	0.040	0.689	0.586	0.0512	0.116	0.116	0.295
V-Beam Roofing and	0.032	0.584	0.497	0.179	0.205	0.205	0.600
Siding, 4-7/8" pitch	0.040	0.730	0.621	0.223	0.255	0.255	0.600
	0.050	0.913	0.776	0.279	0.317	0.317	0.600
Ribbed Siding, 4" pitch	0.032	0.585	0.497	0.0836	0.160	0.175	0.410
	0.040	0.730	0.621	0.104	0.198	0.217	0.410
Ribbed Siding, 8" pitch	0.032	0.518	0.441	0.0648	0.0895	0.235	0.383
	0.040	0.648	0.551	0.0810	0.111	0.289	0.383
V-Beam Roofing and	0.032	0.580	0.494	0.199	0.229	0.229	0.635
Siding, 5-1/3" pitch	0.040	0.726	0.617	0.249	0.285	0.285	0.635
	0.050	0.907	0.771	0.311	0.354	0.354	0.635

Data for these sections are from Table 64, page 93 of "Engineering Data for Aluminum Structures" by The Aluminum Association.

specified herein, the structural performance will be established from the evaluation of load tests. Unless specified otherwise, load tests for steel sheets will be in accordance with AISI Specification for the Design of Cold-Formed Steel Structural Members, and for aluminum sheets, the Aluminum Association Specifications for Aluminum Structures. Testing will consider uplift and loadings applicable to edges and corners as appropriate. Test procedures will be subject to approval by the appropriate headquarters. Load tests on structural performance will meet the strength and deflection requirements in the above referenced publications except as modified herein. Tests will also include an evaluation of the adequacy of watertightness at laps and seams when subject to full dead and live loads or wind loads. Load test results and evaluation data will be submitted for approval.

b. Modified ASTM E 330 test. Underwriters Laboratories, Inc. (UL) 580 test classifies roof systems. It does not determine the load carrying capacity of a particular roof system. The ASTM is developing a standard for an air pressure test that will approximate the load carrying capacity of a roof system. ASTM E330 has been modified by the Corps of Engineers to test flexible metal panels. Until the ASTM standard on SSMR systems is published, the Corps of Engineers' "Test Methods For Structural Performance Of Standing Seam Metal Roof Systems By Uniform Static Air Pressure Difference" (Appendix D) will be used to determine the load carrying capacity of a SSMR system that requires testing. If the SSMR system deviates from the design requirements in this chapter, the SSMR system will be tested in accordance with the Corps of Engineers' test (Appendix D). It is recommended that the specifications require the manufacturer to test each SSMR system once in accordance with with the Corps of Enginners' test. The test may also be required if the building is identified by the user as a critical building and the designer wants to proof test the SSMR system.

c. Component testing. It may be desirable to have the clips and fasteners tested in a laboratory or the field by means of direct-pull tension tests. This would serve to check the connection of clips and fasteners to purlins and subpurlins, and of the fasteners securing the roof purlins to the building's structural system. The test loading will be the design wind uplift pressure increased by the appropriate factor of safety. The components should be tested to the design load and cycled five times. There should be no permanent deformation of the clips or loosening of the screws or other fasteners.

d. Re-roof guidance. For re-roof applications there should be pull out tests to check the tensile strength of the fasteners securing the roof purlins to the existing building's structural system. The test loading will be design wind uplift pressure increased by the factor of safety discussed earlier. The component(s) should be test loaded to the factored design wind uplift pressure and cycled five times. There should be no permanent deformation of the clips or lossening of the screws or other fasteners. The contractor will be responsible for these tests when they are required in the contract documents.

CHAPTER 7

METAL BUILDING SYSTEMS

7-1. Introduction

This chapter prescribes the design criteria for metal building systems.

7-2. Definitions

a. Metal building systems. Metal building systems are buildings which are supplied as a complete building unit. They are to be the product of one metal building supplier. This supplier will be responsible for the design, fabrication, erection, inspection, and quality control of the structure.

b. Standard metal building systems. Standard metal building systems are metal building systems that are designed in accordance with "Low Rise Building Systems Manual" by the Metal Building Manufacturers Association (MBMA). These buildings have an eave height equal to or less than 20 feet, or have rigid spans less than or equal to 80 feet. Typical examples of standard metal building systems include warehouses, pump houses, and servicing facilities.

c. Special purpose metal building systems. Special purpose metal building systems are metal building systems designed by the manufacturer to meet loadings specified by the Government. These buildings have an eave height greater than 20 feet or rigid frame spans greater than 80 feet, or are buildings considered to be special application due to factors other than size, such as use, replacement value of contents, or location. Typical examples may be large gymnasiums, aircraft hangars, maintenance shops, or other large clear span industrial type buildings.

d. Custom metal buildings. Custom metal buildings are metal buildings which are designed by an architect-engineer firm for a specific set of circumstances; and although they may utilize standard parts, they are not designed or fabricated completely by the manufacturer or designated by the manufacturer as a metal building system.

7-3. Design criteria

a. Loadings.

(1) Standard metal building systems. For standard metal building systems, floor load combinations and procedures for developing the design loads will follow the criteria in the MBMA publication "Low Rise Metal Building Systems Manual". The following data will be used in developing design loads:

(a) Dead loads, floor live loads, basic wind speeds, and ground snow loads will be in accordance with TM 5-809-1/AFM 88-3, Chapter 1. Roof live loads will be in accordance with MBMA requirements.

(b) Seismic zone will be obtained from TM 5-809-10/AFM 88-3, Chapter 13. Note that TM 5-809-10/AFM 88-3, Chapter 13 has zones 2A and 2B instead of zone 2 as in MBMA. Zone 2A corresponds to zone 2 in MBMA. For buildings in zone 2B, use $Z = 0.50$ in the lateral force equation for seismic loads in MBMA.

(c) Importance factors for wind and snow loads will be obtained from ASCE 7. Importance factors for seismic loads will be obtained from TM 5-809-10/AFM 88-3, Chapter 13. The building category will be obtained from this document for wind and snow loads and from TM 5-809-10/AFM 88-3, Chapter 13 for seismic loads.

(2) Special purpose metal building systems and custom metal buildings. For special purpose metal buildings systems and custom metal buildings, the load criteria in TM 5-809-1/AFM 88-3, Chapter 1 and TM 5-809-10/AFM 88-3, Chapter 13 will be used in place of the MBMA load criteria.

b. Design of primary framing and structural members. Refer to chapters 3 and 5 for design of steel and aluminum members, respectively.

c. Exterior roof and wall covering. Refer to chapter 6 for design of metal roofing and siding. If masonry walls are used in place of metal wall panels, the masonry must be isolated from the metal building structure.

d. Roof slopes. Refer to discussion of roof slopes in chapter 6.

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CHAPTER 8

MINIMUM STRUCTURAL SYSTEMS FOR WIND LOADS

8-1. Introduction

This chapter prescribes the minimum general criteria for structural systems required to resist wind loads. Refer to TM 5-809-1/AFM 88-3, Chapter 1 for design wind loads. This chapter does not include the criteria and the design requirements for the structural systems needed to resist other horizontal design loadings such as seismic and blast loads.

8-2. Definitions

a. Structural member. The smallest part of a lateral load resisting system that is not part of a connection. Examples would be a beam or a column.

b. Structural element. The smallest complete and separate structural part that reacts as a single unit to resist lateral loads. An element is a moment resisting planar assemblage of members and connections between members. Elements may be oriented either vertically or horizontally. Examples would be a panel of horizontal bracing, a single shear wall, or a planar moment resisting frame.

c. Structural system. A group of elements that react together to give lateral support to the building in all principal directions.

(1) Vertical system. All of the vertical planar elements of the building, such as all the shear walls on the first floor.

(2) Horizontal system. The horizontal planar element or elements at any one level of the building, such as the roof diaphragm of a single floor building.

(3) Total system. All vertical and horizontal structural elements in the building.

d. Structural connection. The parts needed to attach the elements and/or members of the building structure together. Connections may be between members, as the bolts or welds that attach beams to columns, or between elements of systems as the ties attaching a diaphragm to a shear wall.

e. Relative rigidity. This refers to the relative stiffness of the elements of a system. This could be the relative stiffness of all the shear walls in one principal direction, or it could be the relative stiffness of buildings systems, as the diaphragm relative to the vertical system. The relative rigidity influences the manner in which the shear forces from the wind loadings are distributed from the horizontal systems to the vertical systems of the total lateral support system.

f. Deformation compatibility. This refers to the interaction that occurs between nonstructural building elements, such as exterior panels, and the horizontal wind load resisting structural system. This interaction is dependent on the deformations of the structural systems due to design wind loadings.

8-3. Selection of structural system

The goals in the selection of the wind load resisting system are simplicity in the structural framing layout and symmetry in the structural system reaction to design loadings. The selections must consider the need for economy, function, reliability, and deformation compatibility with the architectural and other nonstructural building elements and features. The deflection or drift between adjacent levels of the horizontal structural system (diaphragms) will be limited to 0.005 times the story height.

8-4. Approved structural systems

The minimum structural elements and systems required to resist the design wind loads will be selected from the following:

a. Bearing wall system. A structural system without a complete vertical load carrying space frame. Bearing walls or bracing systems support gravity loads. Shear walls or braced frames resist lateral loads.

b. Building frame system. A structural system with an essentially complete space frame that supports gravity loads. Shear walls or braced frames resist lateral loads.

c. Moment resisting frame system. A structural system with an essentially complete space frame that supports gravity loads. Moment resisting space frames resist lateral loads primarily by flexural action of structural members.

8-5. Alternative structural systems

Alternatives to the approved lateral load resisting systems are permitted if it can be shown that the proposed structural elements and systems selected to resist the design wind loads can be analyzed using a rational structural analysis. Also, the proposed elements and systems will provide a clearly defined and completely interconnected loading path that will transfer the horizontal wind loadings through the system and to the foundation.

8-6. Torsion

Diaphragm torsion may have to be considered in the design depending upon the relative rigidity of the horizontal structural elements to the vertical structural elements. For design guidance, refer to the diaphragm torsional requirements in TM 5-809-10/AFM 88-3, Chap. 13. Regardless of the relative rigidities, the designer must also consider torsion created by unsymmetrical wind loading when the center of loading does not coincide with the center of mass of the structure.

8-7. Connections between elements

Historically, when failures of horizontal load resisting structural systems occur, the connections between the elements of the structural systems have been either the cause of the failure or a major contributing factor in the vast majority of the failure events. The cost of providing the connections between elements needed to resist design wind loads is a very small percentage of the total cost of the structural systems and an even more insignificant percentage of the total building cost. For these reasons, the connections between the major structural elements of the lateral load resisting system (except reinforced concrete frame connections) will resist the load combinations per ASCE 7, except that 2.0 W will be substituted for W in the basic load combinations. (For example, the load combination D + W becomes D + 2W for designing the connections only). However, the connection strength need not exceed the strength of the connected structural member. This criteria is intended for low-rise buildings where the unfactored wind loads are small and do not control the size of the connection. Reinforced concrete moment frame connections will meet the requirements in ACI 318. Seismic design will conform to TM 5-809-10/AFM 88-3, Chapter 13.

APPENDIX A
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TM 809-10-2/AFM 88-3, Ch.13, SecB	Seismic Design Guidelines for Upgrading Exisiting Buildings
TM 5-809-11/AFM 88-3, Ch. 14	Design Criteria for Facilities in Areas Subject to Typhoons and Hurricanes.
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A 307-88	Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength.
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A 514/A 514M-88	High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding
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APPENDIX B

DESIGN OF SLABS ON GRADE

B-1. Introduction

This appendix applies to the design of lightly loaded slabs on grade subject to typical exposure and subgrade conditions and average conditions relative to concrete placement, curing, and shrinkage. This appendix presents alternative design provisions to those set forth in chapter 2 regarding thickness design based on maximum uniform live load. Lightly loaded slabs on grade are those supporting stationary live loads of not more than 400 pounds per square foot, stationary concentrated line (wall) loads of not more than 600 pounds per foot, or vehicle axle loads of not more than 5,000 pounds. Typical exposure conditions refer to interior locations other than airplane hangars where slabs are not subjected to extremes of exposure and warping stresses are limited. Typical subgrade conditions are characterized by sufficient underdrainage to prevent frost penetration, the absence of a wet environment, i.e., volume change due to change in moisture content is limited, and the absence of expansive soils. In addition, typical subgrade conditions are deemed to include only soils classified (according to ASTM D 2487) as either Class ML, any of the S or G groups, or Class CH, CM, or CL having a modulus of subgrade reaction (k) of 100 pounds per cubic inch or greater. Although slabs on grade may be designed to perform satisfactorily on subgrades of lower strength, design for such conditions is beyond the scope of this appendix. Refer to TM 5-809-12/AFM 88-3, Chapter 15 for design of slabs on grade subjected to heavy loads.

B-2. Design requirements

Slabs will be designed for bending stresses due to uniform loads and concentrated loads and for inplane stresses due to subgrade drag. When appropriate for the type facility being designed, slabs will be designed for the effects of warehouse loadings involving aisles, posts and racks, etc. In such instances, particular attention will be given to the design for negative moment in aisles. Refer to PCA Publication "Slab Thickness Design for Industrial Concrete Floors on Grade" and ACI 302.1R for slab thickness design. For typical values of modulus of subgrade reaction (required in computing the slab thickness), refer to TM 5-809-12/AFM 88-3, Chapter 15.

B-3. Subgrade drag (reinforcement)

Subgrade drag force will be calculated on the basis of a coefficient of friction between the slab and subgrade of 1.5 for granular, loose, or soft subgrades susceptible to pronounced indentation during construction. (Subgrade drag force per foot width of slab is 0.5 times the weight of slab per square foot, times the length of slab between control joints, times the coefficient of friction.) Lesser values may be used if a smooth subgrade (plus or minus 0.5 inch deviation from a 10 foot straightedge) can be assured. Neglect transient loads in calculating drag force, but include storage or other long-term live loads. Allowable tensile stress in steel reinforcement may be taken as 0.66 Fy (yield point). Locate reinforcement at 1.5 inches below the surface for control joint spacings of less than 15 feet. Locate reinforcing at one-third the slab thickness below the surface for control joint spacings of 15 feet or more. Refer to PCA publication "Concrete Floors on Ground" and ACI 302.1R for the required area of subgrade drag reinforcement.

B-4. Joints

Control joints should be located at all interruptions in the slab and at 25 foot intervals. If continuous slabs are used, the provision relating to reinforcement for subgrade drag will not apply. Reinforcing for continuously reinforced slabs will be 0.5 percent of the gross cross sectional area of the concrete. Crack control is provided by the presence of the reinforcement; aggregate interlock provides shear resistance across the cracks. Where control joints are used under normal conditions, the following will be considered:

a. To reduce slab curl, erect walls and roof prior to placing the slab.

b. Limit area of hand-placed concrete to 450 square feet between joints.

c. Limit ratio of length to width of pour to 1:1.25.

d. For hot, dry, and other extreme conditions, more stringent limitations will be applied.

APPENDIX C

THICKENED SLAB DESIGN

C-1. Introduction

This appendix provides guidance on the design of thickened slab on grade to support line loads such as partitions. The allowable loads are based on the beams on elastic foundation theory. See Beams On Elastic Foundation, by M. Hetenyi.

C-2. Loading conditions

Partition loads are applied to slabs in the three conditions shown in figure C-1.

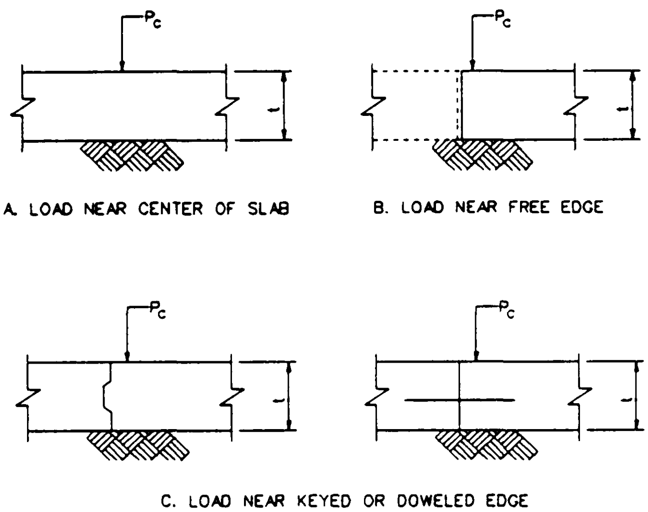

Figure C-1. Conditions of loading.

C-3. Slab design

a. Tabular design loads. Maximum allowable load values, P_c or P_e, in pounds per linear foot that can be supported by various slab thicknesses are given in table C-1. Separate wall footings will be used to support loads exceeding the table values. Values in table C-1 are based on the concrete flexural strength indicated and k = 100 pounds per cubic inch, where k is the modulus of subgrade reaction. Table C-2 may be used to estimate the values of k.

Table C-1. Allowable Slab Line Loads.

Slab Thickness t inches	Pc (plf), Load Near Center of Slab or Near Keyed Joint or Joint with Dowel or Tie Bar	Pe(plf),Load Near Free Edge
4	485	375
5	640	495
6	805	620
7	975	755
8	1,150	890
9	1,330	1,036
10	1,520	1,180

Tabulated loads are based on a modulus of subgrade reaction (k) of 100 pounds per cubic inch and a flexural strength of concrete = 650 psi. The thickness of the slab for other values of k will be computed by multiplying the thickness in Table C-1 by the factor;

$$\sqrt[5]{\frac{100}{k}}$$

Typical values of this factor are:

k (psi)	25	50	100	250	300
Factor	1.3	1.1	1.0	0.83	0.8

For this application, the flexural strength of concrete has been assumed equal to $9\sqrt{f'c}$ where f'c is the specified compressive strength of concrete (psi).

b. Width of thickened slab. The width of thickened areas for slabs on grade which support line loads will be in accordances with figure C-2.

c. Modulus of subgrade reaction. For the design of rigid floor slabs in areas where no previous experience regarding the floor slab performance is available, the modulus of subgrade reaction, k, to be used for design purposes may be determined by soil classification and moisture contents. Table C-2 lists typical values of modulus of subgrade reactions for various soil types and moisture contents ranges.

C-4. Examples

The following are examples of the application of information included in this appendix.

a. Figure C-3. Slab design example 1, provides a way for determining required thickness and width of thickening for a line load located near the center of a slab.

b. Figure C-4. Slab design example 2, provides a means for determining required thickness and width of thickening for a line located near the edge of a slab.

Table C-2. Typical values of modulus of subgrade reactions.

	Module of Subgrade Reactions, k in lb/in^3 for Moisture Contents of							
Types of Mterials	1 to 4%	5 to 8%	9 to 12%	13 to 16%	17 to 20%	21 to 24%	25 to 28%	Over 29%
Silts and clays Liquid limit >50 (OH,CH,MH)	–	175	150	125	100	75	50	25
Silts and clays Liquid limits <50 (OL,CL,ML	–	200	175	150	125	100	75	50
Silty and clayey sands (SM & SH)	300	250	225	200	150	–	–	–
Gravelly sands (SW & SP)	300+	300	250	–	–	–	–	–
Silty and clayey gravels (GM & GC)	300+	300+	300	–	–	–	–	–
Gravel and sand	300+	300+	–	–	–	–	–	–

NOTE: k values shown are typical for materials having dry densities equal to 90 to 95 percent of the maximum CE 55 density. For materials having dry densities less than 90 percent of maximum CE 55 density, values should be reduced by 50 lb/in^3, except that a k of 25 lb/in^3 will be the minimum used for design.

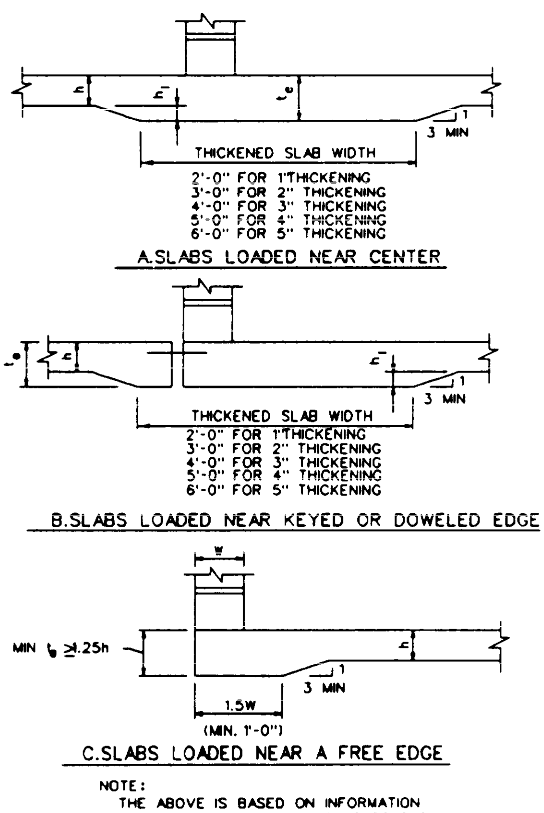

Figure C-2. Width of thickened slabs.

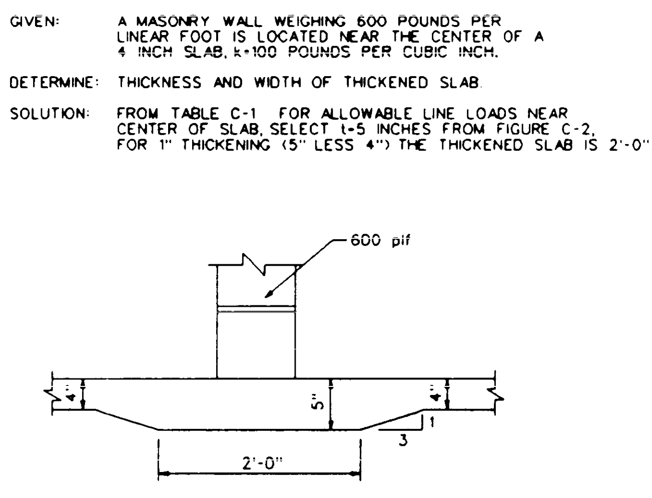

Figure C-3. Slab design Example 1.

GIVEN: AN 8" MASONRY WALL WEIGHING 600 POUNDS PER LINEAR FOOT IS LOCATED NEAR THE EDGE OF A 4 INCH SLAB, k=100 POUNDS PER CUBIC INCH.

DETERMINE: THICKNESS AND WIDTH OF THICKENED SLAB.

SOLUTION: FROM TABLE C-1 FOR ALLOWABLE LINE LOADS NEAR A FREE EDGE, SELECT t=6 INCHES FROM FIGURE C-2, MIN. te=5"; USE 6". FOR W=8", THE THICKENED SLAB WIDTH IS 12" (=1'-0" MIN.)

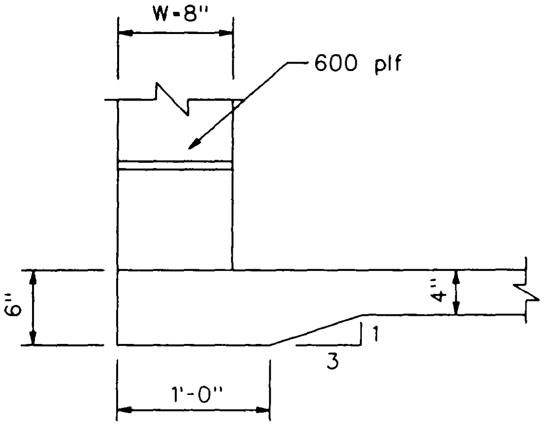

Figure C-4. Slab design Example 2.

APPENDIX D

TEST METHOD FOR STRUCTURAL PERFORMANCE OF STANDING SEAM METAL ROOF SYSTEMS BY UNIFORM STATIC AIR PRESSURE DIFFERENCE

D-1. Scope

a. This test method covers the determination of the structural performance of standing seam metal roof systems under uniform static air pressure differences, using a test chamber.

b. The proper use of this test method requires a knowledge of principles of pressure and deflection measurement.

c. This test method describes the apparatus to be used for applying specific test loads uniformly distributed to a specimen.

d. The values stated in inch-pound units are to be regarded as the standard. The metric equivalents of inch-pound units may be approximate.

e. This test method may involve hazardous materials, operations, and equipment. This test method does not purport to address all of the safety problems associated with its use. It is the responsibility of whomever uses this test method to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

D-2. Descriptions of terms specific to this test method

a. Specimen - the entire assembled unit submitted for test.

b. Test Load - the specified difference in static air pressure (positive or negative) for which the specimen is to be tested, expressed in pounds-force per square foot (or pascals).

c. Ultimate Load - the difference in static air pressure (positive or negative) at which failure of the specimen occurs, expressed in pounds per square foot (or pascals).

d. Permanent Deformation - the permanent displacement from an original position that remains after an applied load has been removed.

e. Failure - the inability to carry additional load, permanent separation of the panels or of any of the component parts or connections or when permanent deformation is sufficient to compromise the weatherability of the roof system.

D-3. Summary of method

This test method consists of sealing the test specimen onto the face of a test chamber, supplying air to or exhausting air from the chamber at the rate required to maintain the test-pressure difference across the specimen, and observing, measuring, and recording the deflections, deformations, and nature of any failures of principal or critical members.

D-4. Significance and use

a. This test method is a standard procedure for determining structural performance under uniform static air pressure difference. This typically is intended to represent the effects of wind loads on building roof surfaces.

b. Design wind pressures will be given in the contract documents. The maximum pressure required in this test will be the design wind uplift pressure multiplied by a factor of safety of 1.65 or 140 psf whichever is the lesser.

D-5. Apparatus

a. The description of the apparatus is general in nature; any equipment capable of performing the test procedure within the allowable tolerances is permitted.

b. Major components

(1) Test chamber - A test chamber or box with an open top upon which the specimen is installed. At least one static pressure tap will be provided to measure the chamber pressure and will be so located that the reading is unaffected by the velocity of the air supply to or from the chamber or any other air movement. The air supply opening into the chamber will be arranged so the air does not impinge directly onto the test specimen with any significant velocity. A means of access into the chamber may be provided to facilitate adjustments and observations after the specimen has been installed. The test chamber or the specimen mounting frame, or both, must not deflect under the test load in such a manner that the performance of the specimen will be affected.

(2) Air system - A controllable blower, compressed air supply, an exhaust system, or reversible controllable blower designed to provide the required maximum air pressure difference across the specimen. The system will provide an essentially constant air pressure difference for the required test period.

(3) Pressure-measuring apparatus - A device to measure the test pressure difference within a tolerance of +/- 2%

(4) Deflection-measuring apparatus - A means of measuring deflections within a tolerance of +/- 0.01 inch (0.25 mm).

(a) Deflections will be measured at the midpoint between supporting roof members. These measurements will be taken along the standing seam and at the midpoint of the panels between standing seams. Deflection of edge members that are part of the roof system will also be measured at similar locations. Additional measurements may be required by the specifier.

(b) When deflections are to be measured, the deflection gages will be installed so that the deflections of the components can be measured without being influenced by possible movements of, or movements within, the specimen or member supports.

D-6. Safety precautions

Take proper precautions to protect the observers in the event of any failure. At the pressures used in this test method, considerable energy and hazard are involved. Do not permit personnel in such chambers during the tests.

D-7. Test specimens

a. Specimens will be of sufficient size to determine the performance of all parts of the system (approximately 10 feet by 20 feet). Conditions of structural support members will be simulated as accurately as possible. All parts of the test specimen will be full size, using the same materials, details and methods of construction and anchorage as used on the actual building. Two specimens are required to perform the test. There will be one specimen representing the corner condition and a second specimen representing the construction in the middle portion of the roof. The second test may be waived by the EOR if the test with members spaced at 5 feet on centers resists the wind pressures for the perimeter areas.

b. Width: Edge seals will not contain structural attachments that restrict deflection of the test panels other than the normal gable conditions.

c. Length: Spacing of the supports will be the actual spacing of the panel spans being evaluated with appropriate panel overhangs, if any, at end supports. The minimum number of spans to be tested is three and the minimum length is 15 feet.

D-8. Calibration

Calibration of manometers and deflection-measuring devices is normally not required, provided the instruments are used at or near their design temperature.

D-9. Required information

In specifying this method the following information will be supplied by the specifying authority:

a. The number of incremental loads and the positive and negative test loads at these increments at which deflection measurements are required (see below).

b. The duration of incremental and maximum loads (see below).

c. The number and location' of required deflection measurements, discussed previously.

D-10. Procedure

a. Fit the specimen upon the chamber opening. Support and secure the specimen by the same number and type of anchors normally used in installing the unit on a building.

b. If air leakage through the test specimen is excessive, tape may be used to cover any cracks and joints through which the leakage is occurring. Tape will not be used when there is a probability that it may significantly restrict differential movement between adjoining members. It is also permissible to cover the entire specimen and mounting panel with a single thickness of polyethylene film no thicker than 6 mils (0.006 in.)(0.152 mm). Panels should be tested in such a manner that 100% of the exposed panel surface has a uniform static air pressure difference applied giving particular attention to insuring that this load is applied between or behind all framing support members. The technique of application is important in order that full load is permitted to be transferred to the specimen and that the membrane does not prevent movement or failure of the specimen. Apply the film loosely with extra folds of material at each corner and at all offsets and recesses. When the load is applied, there will be no fillet caused by tightness of plastic film that will have a significant effect on the results.

c. Install any required deflection-measuring devices at their specified locations. A minimum of 6 points on the load-deflection curve will be obtained.

d. Apply a nominal pressure equal to at least four times the dead weight of the specimen. Use this nominal pressure as the "reference zero load" and record initial readings after the applied load has been applied for 60 seconds and until dial gauges indicate no further increase in deflection. The next load, unless otherwise specified, will be a load equal to one-quarter the design wind uplift load and will be applied for not less than 60 seconds. Thereafter, reduce the pressure difference to no load and then back to "reference zero load". Take readings to determine the permanent deformation or failure. Continue to apply loading in the specified increments reducing loading after each increment back down to no load and then back to "reference zero load" until the design wind uplift load is achieved. After completion of the above portion of the test, reduce the pressure difference to no load and then back to "reference zero load". Multiply the design wind uplift load by the factor of safety. Using one third increments of this value, increase the load in three steps until this factored load is reached. Follow the same procedure as used to test to design load. Hold the test at final load for a minimum of 60 seconds then reduce the load to zero.

If failure occurs prior to the full load, record the load at the time of failure.

D-11. Report

The report will include the following information:

a. Date of the test and the report.

b. Identification of the specimen (manufacturer, source of supply, dimensions, model types, materials, and other pertinent information).

c. Detailed drawings of the specimen, showing dimensioned section profiles, framing location, panel arrangement, installation and spacing of anchorage, and any other pertinent construction details. Any modifications made on the specimen to obtain the reported values will be noted on the drawings.

d. A tabulation of the number of the test load incremnts, the pressure differences exerted across the specimen at these increments, the pertinent deflections at these pressure differences, and permanent deformations at locations specified for each specimen tested.

e. The duration of test loads, including incremental loads.

f. A record of visual observations of performance.

g. When the tests are made to check conformity of the specimen to a particular specification, an identification or description of that specification.

h. A statement that the tests were conducted in accordnce with this test method, or a full description of any deviations from this test method.

i. A statement as to whether or not tape or film, or both, were used to seal against air leakage, and whether in the judgment of the test engineer, the tape or film influenced the results of the test.

j. If several essentially identical specimens of a component are tested, results for all specimens will be reported, each specimen being properly identified particularly with respect to distinguishing features or differing adjustments. A separate drawing for each specimen will not be required if all differences between them are noted on the drawing provided.

k. The test will be performed by an independent testing laboratory or at the manufacturer's laboratory if an independent Registered Professional Engineer witnesses the test. In either case, the test report shall be signed and sealed by the engineer who witnessed the test.

The proponent agency of this publication is the Office of the Chief of Engineers, United States Army. Users are invited to send comments and suggested improvements on DA Form 2028 (Recommended Changes to Publications and Blank Forms) to HQUSACE, (CEMP-ET), WASH DC 20314-1000.

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