An Introduction to Structural Design Criteria for Buildings

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An Introduction to Structural Design Criteria for Buildings

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1. CONCRETE

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

1.2 BASIS FOR DESIGN. The basis for design for buildings and building components constructed of reinforced concrete, prestressed concrete, or plain concrete will be ACI 318, "Building Code Requirements for Structural Concrete and Commentary". Additional provisions for buildings constructed in severe environments, and buildings designed to resist the effects of accidental explosions (blast-resistant construction) are discussed elsewhere. 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, and to ACI standards and committee reports referenced in this publication.

1.3 EARTHQUAKE RESISTANT DESIGN. Concrete structures are to be designed to resist the effects of earthquake ground motions. The additional requirements of TI 809-04, "Seismic Design for Buildings" and FEMA 302, "NEHRP Recommended Provisions for the Seismic Design of New Buildings and Other Structures" will apply.

1.4 DESIGN STRENGTHS. Concrete strengths for various applications and various exposures are listed in Table 1. Use of high strength concrete will be in accordance with ACI Committee 211 Report, "Guide for Selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash," and ACI Committee 363 Report, "State-of-the-Art Report on High Strength Concrete."

Usage	Minimum Strength
Concrete fills.	14 Mpa (2000 psi)
Encasements for utility lines and ducts.	17 Mpa (2500 psi)
Foundation walls, footings and cast-in-place concrete piles.	20 Mpa (3000 psi)
Slabs on grade.	24 Mpa (3500 psi)
Reinforced concrete buildings	20 Mpa (3000 psi)
Precast members (including architectural and structural members and piles).	28 Mpa (4000 psi)
Walls or floors subjected to severe exposure (Severe exposure includes extreme heat or cold and exposure to deicing or other aggressive chemicals.)	20 Mpa (3000 psi)
Concrete deposited under water (tremie concrete).	20 Mpa (3000 psi)
Columns in multistory buildings carrying heavy loads.	28 Mpa (4000 psi)
Reinforced concrete in contact with sea-water, alkaline soils or waters, or other destructive agents.	28 Mpa (4000 psi)
Prestressed concrete construction.	35 Mpa (5000 psi)

Table 1

Minimum Concrete Strength Requirements

1.5 DESIGN CHOICES. The selection of the 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. Designers should take 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 ACI Committee 201 Report, "Guide to Durable Concrete."

1.6 SERVICEABILITY. Buildings must remain serviceable throughout their service life. This means for concrete buildings and concrete structural elements, the concrete must be durable, free from objectionable cracking, and with adequate protection of the reinforcing steel to prevent corrosion. In additions, structural deflections that can damage interior partition walls, ceilings and various architectural features must be kept within acceptable limits.

1.6.1 DURABILITY. Durability of Portland cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment. Causes of concrete deterioration, such as freezing and thawing, aggressive chemical exposure, abrasion, corrosion of steel and other materials embedded in concrete, and chemical reactions of aggregates are described in the ACI Committee 201 Report, "Guide to Durable Concrete". This report also covers various preventive measures to assure durability problems do not occur. The most significant causes of concrete deterioration are freezing and thawing, and corrosion of reinforcing steel.

1.6.1.1 FREEZE-THAW PROTECTION. Concrete made with good aggregates, low water-cement ratio, and air entrainment will have good resistance to cyclic freezing. Air entrained concrete which contains an appropriate distribution of air voids provides good freeze-thaw protection, because when the concrete freezes there is room for any water which has saturated the concrete to expand without causing damage to the concrete. Table 2 provided recommended air contents to prevent freeze-thaw damage.

Average air content %		
Nominal Maximum Aggregate Size	Severe Exposure	Moderate Exposure
10 mm (3/8 inch)	7 1/2	6
12 mm (1/2 inch)	7	5 1/2
19 mm (3/4 inch)	6	5
38 mm (1 1/2 inch)	5	4 1/2
75 mm (3 inch)	4 1/2	3 1/2
150 mm (6 inch)	4	3

Table 2

Recommended Air Contents for Frost-resistant Concrete

(From ACI Committee 201 Report)

1.6.1.2 CORROSION PROTECTION. Corrosion protection is accomplished primarily by providing a sufficient thickness of concrete cover over reinforcing steel and other embedded items. A complete discussion of corrosion causes and preventive measures can be found in the ACI Committee 201 Report, "Guide to Durable Concrete," and in ACI Committee 222 Report, "Corrosion of Metals in Concrete." For normal exposure conditions, or those conditions where the concrete is not exposed to chlorides, the minimum concrete cover protection specified in ACI 318 will be provided. Concrete cover requirements for severe exposure conditions is covered elsewhere in the technical literature.

1.6.2 CRACK CONTROL. Cracking in concrete occurs mainly when volume changes due to drying shrinkage and temperature effects are restrained. Cracking can also occur due to externally applied loads. Cracks indicate a major structural problem, or a serviceability problem. Reinforcing steel exposed to moisture and air can corrode. The corroded steel has a volume several times that of the parent material. Cracking and spalling occurs due to the expansion of the steel as it corrodes. A discussion of the factors that cause cracking in concrete and measures that can be used to control cracking are provided in the ACI Committee 224 Report, "Control of Cracking in Concrete Structures." Cracking can be controlled by providing adequate temperature and shrinkage reinforcement, by reducing steel stresses at service load conditions, and by reducing restraint through the use of joints. Tolerable crack widths for reinforced concrete under various exposure conditions are provided in Table 3.

Exposure Condition	Tolerable Crack Width
Dry air or protective membrane	0.40 mm (0.016 inch)
Humidity, moist air, soil	0.30 mm (0.012 inch)
Deicing chemicals	0.20 mm (0.007 inch)
Sea water and saltwater spray	0.15 mm (0.006 inch)
Water retaining structures	0.10 mm (.004 inch)

Table 3
Tolerable Crack Widths for Reinforced Concrete
(From ACI Committee 224 Report)

1.6.2.1 SHRINKAGE AND TEMPERATURE REINFORCEMENT. To keep cracks widths within acceptable limits for buildings under normal exposure conditions the minimum shrinkage and temperature reinforcement as required by ACI 318 will be provided. Shrinkage and temperature steel requirements for buildings under severe exposure conditions are provided elsewhere in the technical literature.

1.6.2.2 REDUCING STEEL STRESSES UNDER SERVICE LOAD CONDITIONS. Cracking due to service loads can be controlled by limiting the maximum stress in the reinforcing steel, and by providing small diameter bars at close spacing, rather than large size bars at wide spacing. Rules for distributing flexural reinforcement in beams and slabs to control flexural cracking are provided in ACI 318. Suitable distribution of flexural reinforcement in beams and slabs is measured by a z-factor. Z-factors for normal interior and exterior exposure conditions will comply with ACI 318 requirements.

1.6.2.3 JOINTS AND JOINT SEALANTS. The effects of deflection, creep, shrinkage, temperature contraction and expansion, and the need for vibration isolation will all be addressed when determining the location of expansion and contraction joints in concrete buildings. Appropriate allowances for the aforementioned effects will be included in the design; location, details or provisions for required contraction joints, control (weakened-plane) joints, expansions joints, isolation joints, and seismic joints. The location of expansion, contraction, and seismic joints will be shown on the drawings since joints are critical with respect to other design considerations, e.g., configuration of the structural concrete, effects of joints on structural strength and shrinkage cracking, and the appearance of joint lines on exposed concrete surfaces. 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. The following are basic requirements for the more common types. Additional information on joints for concrete buildings can be found in ACI Committee 224.3 Report, "Joints in Concrete

Construction," and the Portland Cement Association Report (PCA), "Building Movements and Joints".

1.6.2.3.1 EXPANSION JOINTS. Expansion joints are seldom needed in buildings less than 200 feet in length, the exception being for brick masonry construction where expansion joints are provided at close intervals. The maximum permitted spacing of expansion joints in brick walls are provided in TI 809-06, “Masonry Structural Design for Buildings”. The maximum length a building can be without expansion joints depends on the temperature change that can occur in the region in which the building is located. In general, expansion joints should be provided in accordance with the following rules:

- 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 to be expected, is not greater than 20 degrees C (36 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 90 meters (300 feet) between joints.

- Where the TD is greater than 20 degrees C (36 degrees F), or where excessive change in atmospheric moisture is likely, expansion joints should be spaced so straight lengths of building measure no more than 60 meters (200 feet) between joints.

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

- Joints for structural or seismic reasons are often located at junctions in L-, T-, or U-shaped buildings.

Expansion joints should extend entirely through the building, completely separating it into independent units. Column footings located at expansion joints need not be cut through unless differential settlements or other foundation movements are anticipated. Expansion joints should be carried down through foundation walls: otherwise the restraining influence of the wall below grade, without a joint, may cause the wall above to crack in spite of its joint. Reinforcement must never pass through an expansion joint. An empirical approach for determining the need for expansion joints is provided in the PCA Report, "Building Movements and Joints".

1.6.2.3.2 CONTROL JOINTS. Control joints are needed to eliminate unsightly cracks in exposed building walls by controlling the location in which cracking due to volume change effects takes place. As a general rule:

- In walls without openings, space control joints at 6-meter (20-foot) intervals; in walls with openings, space at 8-meter (25-foot) intervals.

- Provide a control joint within 3 to 5 meters (10 or 15) feet of a corner.

- Where steel columns are embedded in the walls, provide joints in the plane of the columns.

- If the columns are more than 8 meters (25 feet) apart, provide intermediate joints. Numerous ways have been developed for forming control joints in walls. Whatever method is used, the thickness of the wall section at the joint should be reduced at least 20% by the depth of the joint; and the sum of the depths of the inside and outside grooves should not be less than 50 mm (2 inches).

1.6.2.3.3. 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.

1.6.2.3.4 SEISMIC JOINTS. Buildings that are irregular in plan such as T, L, U, or cruciform shaped buildings can generate high torsional or twisting effects when subjected to earthquake ground motions. These structures would require a three-dimensional analysis for a rigorous determination of stress distribution. Since such analyses are generally not practical, seismic joints are provided to separate various blocks of the structure into regular shaped units that will not exhibit a torsional response. The joints should be of sufficient width to prevent hammering on adjacent blocks during earthquakes, and should be adequately sealed to protect the structure from the environment.

1.6.2.3.5 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.

1.7 LOAD PATH INTEGRITY. Loads must be transferred from their point of application to the foundation. All structural elements and connections along the load path must have sufficient strength, and in the case of seismic resistant structures, sufficient ductility to transfer the loads in a manner that will not impair structural performance. Most load path deficiencies are a result of inadequate connections between precast elements, or between cast-in-place concrete elements and precast elements. Connections are often required to:

- Transfer shear from floor and roof diaphragms to the walls
- Transfer shear from the walls to the foundations
- Transfer shear between individual wall panels
- Transfer tension caused by overturning forces

- Transfer shear, bending, and axial loads between beams and columns and between beams and walls.

Connections between precast elements, or between cast-in-place concrete elements and precast elements, can include the following types of connections:

- Column to foundation
- Column to column
- Beam to column
- Slab to beam
- Beam to girder
- Beam to beam
- Slab to slab
- Wall to foundation
- Slab to wall
- Beam to wall
- Wall to wall

Details for these various types of connections can be found in the Prestressed Concrete Institute (PCI) Technical Report No. 2, "Connections for Precast Prestressed Concrete Buildings".

1.7.1 SHEAR CONNECTIONS. Shear connections are classified as either "wet" or "dry". Wet connections use reinforced or unreinforced cast-in-place concrete to form the junction between members. Dry connections utilize a mechanical anchor, such as bolts or welded metal, to transfer load. Wet and dry connections use shear-friction resistance to transfer forces. In wet connections the reinforcing steel placed across the potential failure plane provides the clamping force needed to provide the shear-friction resistance. The most common type of dry connection involves embedded plates or other structural steel shapes that are anchored to the concrete by welded studs, anchor bolts, or expansion anchors. The embedded plates or structural steel shapes embedded in each of the concrete elements to be connected, are then connected themselves by weldments or by bolting. Contractors prefer the “dry” type connections because they are the easiest types to construct. The "wet" type connectors however are usually the best performers, especially under cyclic loading conditions such as occur during earthquakes.

1.7.2 EMBEDDED BOLT AND HEADED STUD ANCHORS. Embedded anchor bolts and headed studs are commonly used to transfer shear and tension loads between cast-in-place concrete and precast concrete members and between cast-in-place concrete and structural steel shapes. Anchor connections should be designed and detailed to assure connection failure will be initiated by failure of the anchor steel rather than by failure of the surrounding concrete. The design of anchor bolt connectors will be based on the requirements of FEMA 302, Paragraph 9.2, "Bolts and Headed Anchors in Concrete." Additional information regarding the use of headed anchor bolts for anchorage can be found in the American Institute of Steel Construction (AISC) Engineering Journal, Second Quarter / 1983 Report, "Design of Headed Anchor Bolts," and in the Prestressed Concrete Institute (PCI) Design Handbook. Where strength design is used, the required strength, and the design of the anchors will be in accordance with FEMA 302. When allowable stress design (ASD) is used, the allowable service load for headed anchors in shear or tension, assuming the anchor bolts conform to ASTM 307 or an approved equivalent can be assumed equal to that indicated in Table 4. For ASD design, when anchors are subject to combined shear and tension, the following relationship will be used:

$$(P_s/P_t)^{5/3} + (V_s/V_t)^{5/3} \leq 1$$

where:

P_s = applied tension service load

P_t = allowable tension service load from Table 4

V_s = applied shear service load

V_t = allowable shear service load from Table 4

The allowable service loads in tension and shear specified in Table 4 are for the edge distances and spacing specified. The edge distance and spacing can be reduced to 50 percent of the values specified with an equal reduction in allowable service load. Where edge distance and spacing are reduced less than 50 percent, the allowable service load will be determined by linear interpolation. Increase of the values in Table 4 by one-third is permitted for load cases involving wind or earthquake. Where special inspection is provided for the installation of anchors, a 100 percent increase in the allowable tension values of Table 4 is permitted. No increase in shear value is permitted.

1.7.3 EXPANSION ANCHORS. Expansion anchors will be designed in accordance with the provisions of ACI Committee 449 Report, "Concrete Nuclear Structures," Appendix B, "Steel Embedments." The engineer will review expansion anchor design features, failure modes, test results and installation procedures prior to selecting a specific expansion anchor for an application. Expansion anchors will not be used to resist vibratory loads in tension zones of concrete members unless tests are conducted to verify the adequacy of the specific anchor and application. Expansion anchors will not be installed in concrete where there are obvious signs or cracking, or deterioration.

Minimum Concrete Strength (f'_c)

Bolt Dia.	Minimum Embed.	Edge Distance	Spacing	20 MPa (2500psi)	20 Mpa (2500psi)	25 MPa (3000psi)	25 MPa (3000psi)	30 MPa (4000psi)	30 MPa (4000psi)
				Tension	Shear	Tension	Shear	Tension	Shear
6.4 mm (1/4")	63.5 mm (2-1/2")	38.1 m (1-1/2")	76.2 mm (3")	890 N (200#)	2225 N (500#)	890 N (200#)	2125 N (500#)	390 N (200#)	2225 N (500#)
9.5 mm (3/8")	76.2 mm (3")	57.2 m (2-1/4")	114.3mm (4-1/2")	2225 N (500#)	4890 N (1,100#)	2225 N (500#)	4890 N (1,100#)	2225 N (500#)	4890 N (1,100#)
12.7 mm (1/2")	101.6 mm (4")	76.2 mm (3")	152.4mm (6")	4225 N (950#)	5560 N (1,250#)	4225 N (950#)	5560 N (1,250#)	4225 N (950#)	5560 N (1,250#)
12.7 mm (1/2")	101.6 mm (4")	127 mm (5")	152.4mm (6")	6450 N (1,450#)	7120 N (1,600#)	6670 N (1,500#)	7340 N (1,650#)	6890 N (1,550#)	7785 N (1,750#)
15.9 mm (5/8")	114.3 mm (4-1/2")	95.3 mm (3-3/4")	190.5mm (7-1/2")	6670 N (1,500#)	12,230 N (2,750#)	6670 N (1,500#)	12,270 N (2,750#)	6670 N (1,500#)	12,270 N (2,750#)
15.9 mm (5/8")	114.3 mm (4-1/2")	158.8m m (6-1/4")	190.5mm (7-1/2")	9460 N (2,125#)	13,120 N (2,950#)	9785 N (2,200#)	13,340 N (3,000#)	10,670 N (2,400#)	13,570 N (3,050#)
19.1 mm (3/4")	127.0 mm (5")	114.3m m (4-1/2")	228.6mm (9")	10,000 N (2,250#)	14,460 N (3,250#)	10,000 N (2,250#)	15,835 N (3,560#)	10,000 N (2,250#)	15,835 N (3,560#)
19.1 mm (3/4")	127.0 mm (5")	190.5m m (7-1/2")	228.6mm (9")	12,565 N (2,825#)	19,015 N (4,275#)	13,120 N (2,950#)	19,130 N (4,300#)	14,230 N (3,200#)	19,570 N (4,400#)
22.2 mm (7/8")	152.4 mm (6")	133.4m m (5-1/4")	266.7mm (10-1/2")	11,300 N (2,550#)	16,460 N (3,700#)	11,300 N (2,550#)	18,015 N (4,050#)	11,300 N (2,550#)	18,015 N (4,050#)
25.4 mm (1")	177.8 mm (7")	152.4mm (6")	304.8mm (12")	13,570 N (3,050#)	18,350 N (4,125#)	14,460 N (3,250#)	20,020 N (4,500#)	16,235 N (3,650#)	23,575 N (5,300#)
28.6 mm (1 1/8")	203.2 mm (8")	171.5mm (6-3/4")	342.9mm (13-1/2")	15,120 N (3,400#)	21,130 N (4,750#)	15,120 N (3,400#)	21,130 N (4,750#)	15,120 N (3,400#)	21,130 N (4,750#)
31.8 mm (1 1/4")	228.6 mm (9")	190.5mm (7-1/2")	381.0mm (15")	17,790 N (4,000#)	25,800 N (5,800#)	17,790 N (4,000#)	25,800 N (5,800#)	17,790 N (4,000#)	25,800 N (5,800#)

Table 4

Allowable Service Load on Embedded Bolts

(Adapted from the International Building Code (IBC) - Final Draft, Table 1912.2)

1.7.4 ADHESIVE (CHEMICAL) ANCHORS. Adhesive anchors consist of a threaded rod installed in a hole drilled in hardened concrete and filled with a two-component epoxy, polyester, or vinylester resin adhesive. The hole is about 3 mm (1/8") larger than the bolt diameter. Adhesive anchors should not be used in structural elements that are required to be fire resistant. They should not be installed in wet or damp conditions or in concrete where there are obvious signs of cracking, or deterioration. No specific design codes are available for adhesive anchors. Therefore, design should be based on the manufacture recommendations, and testing should be required to assure the installed anchors meet strength requirements. Additional guidance on adhesive anchors can be found in the ACI Paper entitled "Bond Stress Model for Design of Adhesive Anchors", by Cook, R.A., Doerr, G.T., and Klingner, R.E., ACI Journal / Sept-Oct 1993.

1.7.5 OTHER STEEL EMBEDMENTS. Other steel embedments will be designed in accordance with the provisions of ACI Committee 449 Report, "Concrete Nuclear Structures," Appendix B, "Steel Embedments."

1.7.6 SPECIAL CONSIDERATIONS.

1.7.6.1 SHEAR TRANSFER. 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.

1.7.6.2 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.

1.7.6.3 MECHANICAL ANCHORAGE OF REINFORCING STEEL. Mechanical connections are permitted for reinforcing steel in accordance with the provisions of ACI 318. There are many applications that make the use of mechanical connections feasible or more practical. Information on the types of mechanical connectors, their use, and design requirements can be found in the ACI Committee 439 Report, "Mechanical Connections of Reinforcing Bars".

1.8. DETAILING REQUIREMENTS. Details and detailing of concrete reinforcement will conform to ACI 315, "Details and Detailing of Concrete Reinforcement". Engineering and placing drawings for reinforced concrete structures will conform to ACI 315R, "Manual of Engineering and Placing Drawings for Reinforced Structures". For seismic areas, the design and details will conform to TI 809-04, "Seismic Design for Buildings".

1.9. SPECIAL INSPECTIONS.

- Periodic special inspection during and upon completion of the placement of reinforcing steel in intermediate moment frames, in special moment frames, and in shear walls is required.

- Continuous special inspection during the welding of reinforcing steel for structural members is required.

- Periodic special inspection during and upon completion of the placement concrete in intermediate moment frames, in special moment frames, and in shear walls is required.

- Periodic special inspection is required during the placement and after completion of placement of prestressing steel, and continuous special inspection is required during all stressing and grouting operations and during the placement of concrete.

- Periodic special inspection is required during the placement of anchor bolts, expansion anchors, and chemically grouted anchors to verify the anchor system is in conformance with approved plans and specifications.

2. MASONRY

2.1 INTRODUCTION. Design guidance for reinforced masonry structures is provided. Plain (unreinforced) masonry design and design using "empirical" methods are generally not permitted.

2.2 BASIS FOR DESIGN. Reinforced masonry will be designed by allowable stress design (ASD) methods in accordance with TI 809-06, "Masonry Structural Design for Buildings. Loads will be in accordance with Chapter 1, Paragraph 7. Load combinations will be in accordance with Chapter 1, Paragraph 8. Special design and detailing requirements for masonry structures subjected to earthquake ground motions are provided in FEMA 302, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and in TI 809-04, "Seismic Design for Buildings". For buildings in Seismic Design Categories D, E, and F, shear keys at the base of masonry walls may be necessary. Designers must be aware of, and comply with all seismic design and detailing requirements.

2.3 SERVICEABILITY. Detailing of masonry structures to prevent efflorescence, and to prevent cracking due to shrinkage and temperature movements, should be in accordance with recommendations in TI 809-06. Large expanses of roof deck with supporting systems rigidly attached to masonry walls, pilasters, or columns can result in cracking due to thermal changes that take place during construction. Designers should therefore include provisions in the design to accommodate thermal movement both during and after construction. Control joints and expansion joints are used to control cracking due to shrinkage and thermal movement. The joints should permit movement but have sufficient strength to resist required loads. Joints should be weather tight when located in exterior walls. Joint location and detailing requirements are provided in TI 809-06.

2.4 LOAD PATH INTEGRITY. Information on the design and detailing of connections between shear walls (vertical resisting elements) and floor and roof diaphragms (horizontal resisting elements) are provided in TI 809-04, TI 809-06, and FEMA 302. The design of headed anchor bolts and bent-bar anchors embedded in masonry, and used to connect diaphragms and structural members to masonry, will be in accordance with FEMA 302, Paragraph 11.3.12, "Headed and Bent-Bar Anchor Bolts".

2.5 SPECIAL INSPECTIONS.

- Periodic special inspection during the preparation of mortar, the laying of masonry units is required.

- Periodic inspection of vertical, horizontal and bond beam corner reinforcement is required.

- Continuous special inspection during the welding of reinforcement, grouting, consolidation, reconsolidation and placement of anchors (bent bar or anchor bolts) is required.

3. METAL BUILDINGS

3.1 INTRODUCTION. A metal building system is an engineered product furnished by metal building manufacturers. The metal building can be selected from a catalogue of standard designs or can be a custom design. Metal building systems, in general, consist of:

- rigid frames which act as the primary vertical load carrying system and as a lateral force system in the transverse direction,
- rod or angle x-bracing for the roof truss diaphragm and for vertical lateral bracing in the longitudinal direction,
- girts to support wall system cladding and resist wind loads, and
- roof purlins to support a standing seam metal roof system and roof live loads, and an exterior cladding system.

Many variations in framing systems are possible, and many different wall-cladding systems can be used including metal panels, brick veneer, masonry, and precast concrete wall panels.

3.2 METAL BUILDING OPTIMIZATION. The metal building system will be optimized, based on ASCE 7 load and load combination requirements, to provide the lightest weight structural system possible. The designer must determine if additional requirements related to displacement, drift, durability, and redundancy should be written into the specifications to assure the final design will satisfy short and long term performance goals including function, durability, serviceability, and future expansion needs. The major optimization occurs with respect to the rigid frames. These frames are constructed of relatively thin plates that are welded together. The frames use tapered webs with increased depth in the areas of high moments. Web thickness and flange size are varied as needed. The rigid frame members are designed with bolted end connections for easy field assembly. Most often the flanges are weld connected to the web on only one side to reduce fabrication costs. The secondary members, including girts and purlins. are cold-formed with a high strength to weight ratio.

3.3 BASIS FOR DESIGN.

3.3.1 CRITERIA. Specifications for metal building systems usually require that design be in accordance with the Metal Building Manufacturers Association (MBMA), "Low Rise Building Systems Manual," except that design loads and load combinations will be in accordance with TI 809-01, "Load Assumptions for Buildings." Secondary members (purlins, girts, etc.) are normally light gage cold-formed sections for which design is governed by TI 809-07, "Design of Load Bearing Cold-Formed Steel Systems and Masonry Veneer / Steel Stud walls".

3.3.2 ROOFING. Roofing will comply with the requirements of TI 809-29, "Structural Considerations for Metal Roofing."

3.3.3 CERTIFICATION. American Institute of Steel Construction, Inc. (AISC) certification is required of all metal building system manufacturers. However, this requirement may be waived for small storage type buildings with areas less than 140 square meters (1500 square feet).

3.4 STRENGTH AND SERVICEABILITY ISSUES.

3.4.1 GENERAL. Typically metal building systems have the minimum strength required, and use high strength materials to keep strength to weight ratios at a maximum. This approach to design under certain conditions can lead to serviceability problems.

3.4.2 LATERAL DRIFT. Special attention should be paid to building drift under wind and seismic loading conditions. The maximum allowable drift will depend on the type of exterior cladding. For rigid claddings, such as precast concrete, brick, or block masonry, the maximum drift should be limited to h/600. This is much lower than the h/60 limit commonly accepted for buildings with flexible metal cladding.

3.4.3 EARTHQUAKE LOADINGS. Metal building systems with heavyweight cladding must have suitable lateral force resisting systems to resist the inertial forces generated by the cladding during earthquake ground motions. The usual tension-only type bracing conventionally used in metal building construction will not be adequate when heavyweight cladding is used. If rigid cladding is used, the roof diaphragm must also have sufficient stiffness to limit out-of-plane wall displacements so that the cladding will not fail when the building is subjected to earthquake ground motions.

3.4.4 MECHANICAL EQUIPMENT LOADS. The roof purlin system commonly provided with metal building systems would not have the capacity to support hanging mechanical HVAC units or rooftop units. Where the roof is required to support such units, special framing must be provided.

3.4.5 ROOF IN-PLANE LOAD RESISTANCE. Metal building systems are commonly constructed with a standing seam metal roof. Standing seam metal roof systems are incapable of resisting in-plane loads due to wind and earthquake forces. Therefore, a separate horizontal bracing system is required.

3.4.6 SERVICEABILITY GUIDANCE. Metal building systems must meet the same serviceability requirements specified for steel framed buildings. Additional guidance on serviceability can be found in the American Institute of Steel Construction, Inc. (AISC) Steel Design Guide Series 3, "Serviceability Design Considerations for Low Rise Buildings".

3.5 DESIGN RESPONSIBILITY. Performance specifications are generally used to obtain metal building systems. A professional engineer representing the owner will specify the metal building system including all framing elements, roofing system, exterior cladding, interior partition walls, and architectural finishes. The same engineer will specify which design codes, loads, and load combinations are to be considered in the design. Unusual loads such as unbalanced snow loads, and concentrated roof loads must be clearly defined. If future expansion is required, this must also be conveyed so end bays can be designed without intermediate columns. Architectural requirements such as "R" factors for insulation should also be included in the metal building specification. Metal building manufacturer's typically design the building and furnish plans and specifications for building construction. The building manufacturer's design must include all framing elements and their connections, and all exterior wall and cladding systems including openings, framing around openings, and connections. All bays where roof and wall bracing is to be installed should be identified. All interior walls including connections, or separations, from the building frame system should be detailed on the contract drawings. The foundation design will be provided with the contract documents in order to provide a basis for bid. The foundation design will be reviewed after the building manufacturer submits the final design. The final design submitted by the manufacturer should describe all loads, load combinations, and foundation reactions. With the use of gable bent type rigid frames, the foundation must be designed to resist lateral spreading forces. The lateral force resisting anchors are usually hairpin type reinforcing bars embedded in the slab-on-grade. When spreading forces are large, direct tension ties between exterior footings may be necessary. The foundation designer must carefully design and detail all foundation anchor systems to make sure they can transfer loads to the foundation, and make sure they do not intercept slab-on-grade control joints or otherwise interfere with other building features. The professional engineer representing the owner should track the building though the metal building system design review, shop drawing review, and construction process to assure the metal building system actually supplied and erected on the site meets all design requirements. It may be required that certain features of the project, such as foundations, cladding, and connections be redesigned during the metal building system design review phase.

3.6 SPECIAL INSPECTIONS.

- Periodic special inspections will be provided during all welding of elements of the lateral force resisting system.
- Periodic special inspections will be provided for bolting, anchoring, and other components of the lateral force resisting systems including struts, braces, and hold-downs.
- Periodic special inspections will be provided during the erection and fastening of exterior cladding to the metal building framing system.

4. SLABS ON GRADE

4.1 INTRODUCTION. This section provides design and construction guidance for industrial type (non-residential) lightly loaded slabs on grade. Industrial type slabs refer to those slabs that are reinforced to minimize the number of crack control joints (maximize joint spacing), Guidance for unreinforced residential type concrete slabs, which utilize crack control joints spaced at frequent intervals to minimize cracking, can be found in the PCA Publication, "Concrete Floors on Ground." Lightly loaded slabs on grade are those supporting stationary live loads of not more than 20 kPa (400 pounds per square foot), stationary concentrated line (wall) loads of not more than 15 kPa (300 pounds per foot), or vehicle axle loads of not more than 2275 kg (5000 pounds). The guidance is applicable to usual exposure conditions meaning interior locations other than airplane hangars where slabs are not subject to extreme climatic changes, and to typical subgrade conditions 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, "Classification of Soils for Engineering Purposes," 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 2.75 kg per cubic centimeter (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 manual. Refer to TI 809-27, "Concrete Floor Slabs on Grade Subjected to Heavy Loads," for the design of slabs on grade subjected to heavy loads, and to TI 809-28, "Design and Construction of Reinforced Ribbed Mat Slab" for the design of a commonly used slab-on-grade system for resisting potential foundation movements at sites containing expansive soils. Additional information on the design of slabs on grade, primarily industrial floors, can be found in ACI Committee 360 Report, “Design of Slabs on Grade.”

4.2 BASIS FOR DESIGN.

4.2.1 GENERAL. Slabs-on-grade will be designed for bending stresses due to uniform loads and concentrated loads and for in-plane stresses due to drying shrinkage and subgrade drag resistance. 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.

4.2.2 GUIDANCE. Proper construction methods, workmanship, and concrete mix proportioning will follow the guidelines of ACI Committee 302 Report, "Guide to Concrete Floor and Slab Construction". Slabs are required to have a minimum thickness of 100 mm (4 inches). The following thickness for maximum uniform design live loads will be used provided the modulus of subgrade reaction (k) is at least 2.75 kg per cubic centimeter (100 pounds per cubic inch).

Thickness of Slab	Maximum Uniform Design Live Load
100 mm (4”)	7 Kpa (150 psf)
150 mm (5”)	12 Kpa (250 psf)
200 mm (6”)	20 Kpa (400 psf)

Unless otherwise specified above, the correct slab thickness will be determined in accordance with the Portland Cement Association (PCA) Publication entitled, "Slab Thickness Design for Industrial Concrete Floors on Grade." In the PCA design process compressive strength is converted to modulus of rupture, which is then reduced by a factor of safety to obtain the maximum allowable flexural tensile stress. The maximum allowable flexural tensile stress is then used to find the required slab thickness. For typical values of modulus of subgrade reaction refer to TI 809-27, "Concrete Floor Slabs on Grade Subjected to Heavy Loads." When wall loads exceed 15 kPa (300 lb./foot), slabs-on grade will be thickened in accordance with the provisions of TI 809-27.

4.3 SERVICEABILITY.

4.3.1 GENERAL. CRACKING, WARPING, AND CURLING CAN IMPAIR SLAB-ON-GRADE SERVICEABILITY. These problems are directly attributable to drying shrinkage. Cracking can be controlled by minimizing drying shrinkage, by providing adequate crack control and isolation joints, and through the use of reinforcing steel. Water penetrating the slab is a common serviceability problem that can be cured by proper drainage and by the use of vapor barriers.

4.3.2 MINIMIZING DRYING SHRINKAGE. Cracking in slabs generally results from drying shrinkage and restraint caused by friction between the slab and subgrade. Curling and warping occur due to differential shrinkage when the top of the slab dries to lower moisture content than the bottom of the slab. Drying shrinkage, curling, and warping can be reduced by using less water in the concrete. Ways to reduce water content include using the largest maximum sized aggregate (MSA), using an MSA equal to 1/3 the slab thickness, and by using coarse sand. Water content can also be reduced by using coarser ground cement and cement with a low C3A content. On large and critical slab-on-grade projects the designer should request by specification that shrinkage tests be made of several concrete mixes to obtain a mix with the lowest drying shrinkage potential.

4.3.3 CONTROLLING CRACKING THROUGH CONTROL AND ISOLATION JOINTS. Control and isolation joints can be used to minimize cracking and to force cracking to occur at joint locations. Designers should attempt to minimize the number of joints occurring in the slab. However, in most instances, the maximum slab area bound by crack control joints should not exceed 60 square meters (625 square feet), and distance between crack control joints should not exceed 7.5 meters (25 feet). The length/width ratio of panels bounded by joints should be as near 1.0 as possible and should not exceed 1.25. In localities where extreme conditions of heat or dryness tend to produce excessive shrinkage, the maximum area and joint spacing may need to be decreased. Crack control joints may also be construction joints. Joints in the vicinity of column pedestals will be placed at column centerline, with diamond shaped or circular 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 1), cannot be avoided, two No. 13 bars, 1.25 meters long (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 100 mm (4 inches) apart starting at 50 mm (2 inches) from the edge of slab. Except for openings of less than 300 mm by 300 mm (12 inches by 12 inches), corners of openings and reentrant corners in slabs will be reinforced with two No. 13 bars, 1.25 meters long (two No. 4 bars, 4 feet long), placed diagonally to the corner.

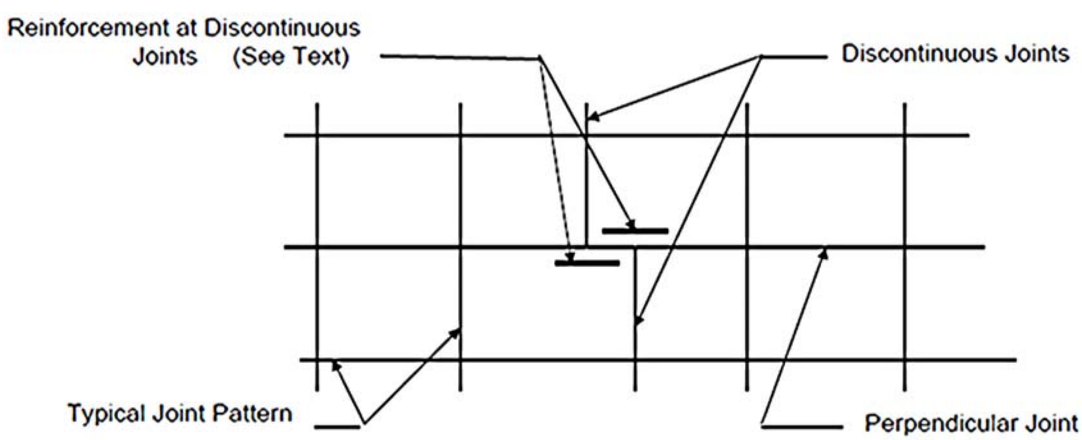
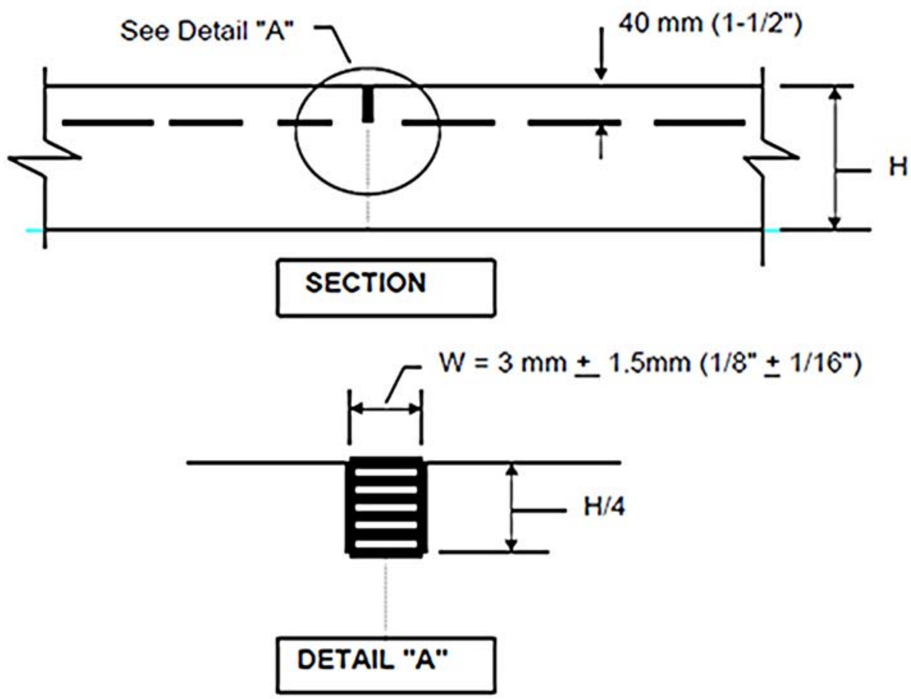


Figure 1

Discontinuous joints

4.3.3.1 CONTROL JOINTS. Control joints form a weakened plane to direct cracking to preselected locations. Sawed control joints will be cut to one-fourth depth of slab thickness (H). Details of control joints are shown in Figure 2. 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.



NOTES
1. Concrete cover of 40 mm (1-1/2 inches) will be provided over reinforcement.
2. One-half of the welded wire mesh reinforcement (alternate wires) will be interrupted within 50 mm (2 inches) of each side of slab control joints

Figure 2

Control joints

4.3.3.2 ISOLATION JOINTS. Isolation joints form a separation of 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 interior and exterior foundation walls and column pedestals, isolation joints will ordinarily be a strip of 15-kg (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, or extreme temperature changes, and where isolation from vibrations of machinery and equipment foundations is required, joint filler 10 mm (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 3.

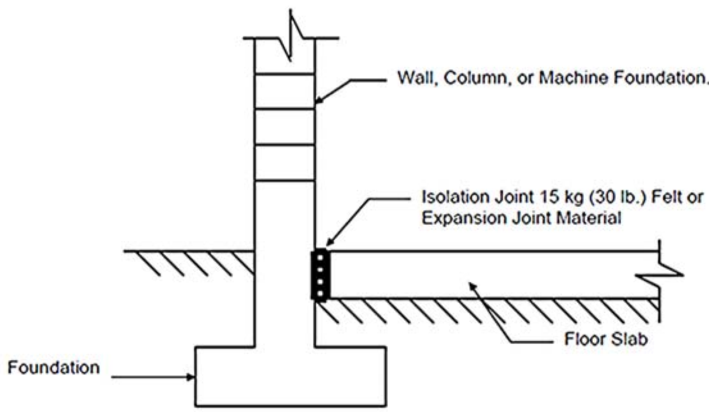
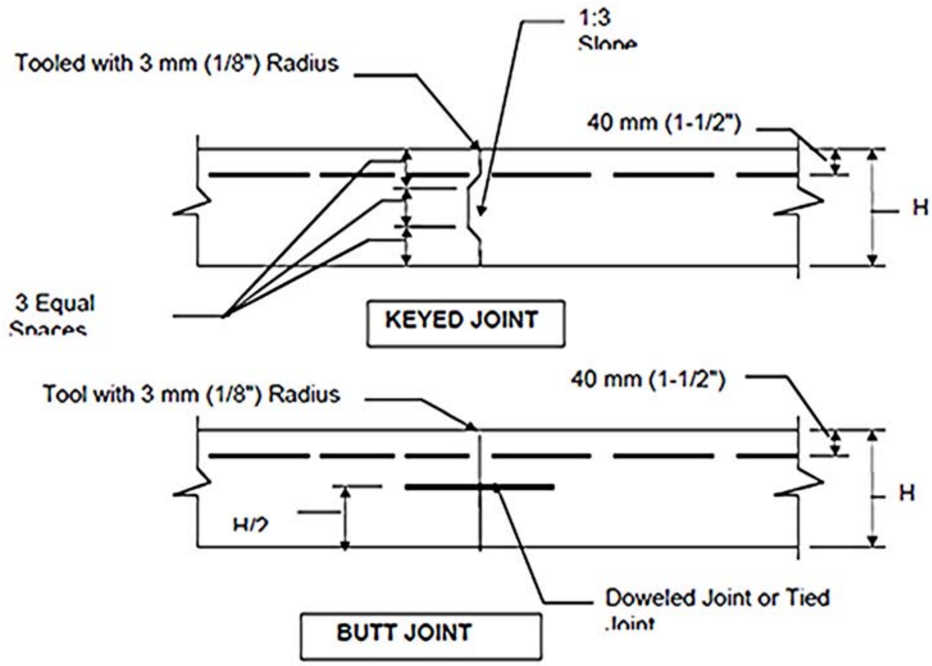


Figure 3
Isolation joints

4.3.3.3 CONSTRUCTION JOINTS. Construction joints are used to allow separate concrete placement. Construction joints should be kept to a minimum, and should generally be in conformity with a predetermined joint layout. Construction joints will have dowels, or keys to provide shear transfer. Dowel size and spacing should be in accordance with ACI Committee 302 Report, "Guide to Concrete Floor and Slab Construction." Formed-keyed joints will only be used in slabs having a thickness of 150mm (6 inches) or more. In order to accommodate keyed joints in 100 mm (4-inch) thick slabs it is acceptable to taper the slab so the slab at joint locations is 150 mm (6-inches) thick. The taper should begin 1 meter (3 feet) each side of the joint. Preformed keys left in place may be used for 100 mm (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 4.



NOTES

1. Keyed joints or doweled joints should align with and function as a control joint or expansion joint. For doweled joints use a 20 mm (3/4") diameter x 400 mm (16 ") long bar and lubricate one half the bar. Dowels should be spaced at 400 mm (16" o.c.) for slab thickness 150 mm (6 inches) or less, and 300 mm (12 inch) spacing for slab thickness greater than 150 mm (6 inches).

2. Tied butt joints constructed with deformed rather than smooth bars will be used when the construction joint is not at a planned control or expansion joint location. This type of joint restrains movement. Use 13 mm (1/2") diameter x 750 mm (30 ") long deformed tie bars spaced at 750 mm (30 ") on centers.

3. Concrete cover of 40 mm (1-1/2 inches) will be provided over reinforcement.

4. Welded wire reinforcement (sheets not rolls) will be stopped 50 mm (2 inches) of each side of planned control or expansion joint locations. Welded wire reinforcement will be continuous through tied butt joints.

Figure 4

Construction joints

4.3.3.4 SLAB-ON-GRADE REINFORCEMENT. Slab-on-grade shrinkage reinforcement will be located 40 mm (1-1/2 inches) from the top of the slab in order to restrain shrinkage and reduce curling. Maximum spacing of reinforcing bars should not exceed three times the slab thickness. For plain wire reinforcement the spacing should not be more than 350 mm (14 inches) longitudinally and 350 mm (14 inches) transversely. The percentage of steel determined should not be less than 0.15 percent. Wire mesh reinforcements meeting the 0.15 percent steel requirement, for various slab thickness, is provided in Table 5. Deformed welded wire fabric in flat sheets, or deformed reinforcing bars will be used. The positioning of the steel in the slab is critical for proper crack control. Reinforcing steel will be supported on chairs and every precaution taken to assure the reinforcing bars are positioned, as intended after construction is complete.

Slab Thickness	Wire Mesh Reinforcement
100 mm (4-inch)	305 x 305 - MW 48.4 x MW 48.4 (12 x 12 - W 7.4 x W 7.5)
125 mm (5-inch)	305 x 305 - MW58.1 x MW 58.1 (12 x 12 - W9 x W 9)
150 mm (6-inch)	305 x 305 - MW 71.0 x MW 71.0 (12 x 12 - W11 x W 11)

Table 5

Minimum slab-on-grade reinforcement requirements

Suppliers because of the large bar sizes and wide spacing may not normally stock the wire mesh reinforcement sizes indicated in Table 5-1. The large bar sizes are desirable to prevent bending of the steel and provide adequate stiffness to keep the steel in the upper half of the slab during concrete placement. For smaller jobs [less than 7000 square meters (75,000 square feet) in those cases when the wire mesh sizes listed in Table 1 are unavailable, wire mesh spacing with reduced bar diameters and closer spacing may be used provided minimum steel requirements are met.

4.3.3.5 VAPOR BARRIERS. High levels of moisture in the subgrade increase slab curling. If the subgrade can become moist because of ground water an impermeable vapor barrier should be provided. The minimum thickness of the vapor barrier should be 1.3 mm (50 mil) and be covered with a 150 mm (6 inches) of crushed stone topped with a 13 mm (1/2 inch) thick layer of sand. The advantage of the 165 mm (6-1/2 inch) stone/sand cover over the vapor barrier is that the vapor barrier will not be punctured nor will the fill material be easily displaced as construction equipment is driven over the stone and sand cover (Ytterberg, 1987). If the only purpose of the vapor barrier is to reduce friction between the slab and subgrade in order to reduce drag, then a polyethylene slip sheet can be placed directly under the slab provided holes are punched in the polyethylene to allow water to leave the bottom of the slab before final set occurs in the concrete.

4.4 SHINKAGE COMPENSATING CONCRETE. Edge curling and shrinkage cracking can be reduced through the use of shrinking compensating concrete that is properly reinforced. The ACI Committee 223 Report, "Standard Practice for the Use of Shrinkage-Compensating Concrete" recommends that the area of steel reinforcement be greater than or equal to 0.15 percent of the cross-sectional area of the concrete and less than or equal to 0.20 percent, and that it be located in the upper half of the slab.

4.5 POST-TENSIONED SLABS. Post-tensioning of slabs-on-grade can be useful where it is desirable to eliminate joints, design the slab-on-grade to resist foundation movements at sites containing expansive soils, or to provide the slab-on-grade system with the capacity to span over weak subgrade areas. The Post-Tensioning Institute (PTI) Reports, "Design and Construction of Slabs-on-Ground," and "Post-Tensioned Commercial and Industrial Floors" can be used as a basis for design of post-tensioned slabs.

4.6 SPECIAL TESTS AND SPECIAL INSPECTIONS. In addition to the usual concrete testing, shrinkage testing of concrete mixtures to minimize shrinkage potential, and modulus of rupture testing of concrete mixtures to maximize the ability of the concrete to resist tensile stress form shrinkage and load induced flexure, should be considered.

5. STEEL STRUCTURES

5.1 INTRODUCTION. This section describes recommended criteria for the design of structural steel, openweb joists, and cold-formed steel structural members in buildings.

5.2 GENERAL. Structural framing systems and elements of buildings will be designed in accordance with the accepted industry standards described 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.

5.3 BASIS FOR DESIGN.

5.3.1 STRUCTURAL STEEL CONSTRUCTION. The design, fabrication and erection of structural steel for buildings and structures will be in accordance with either the AISC Load and Resistance Factor Design (AISC-LRFD) or AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD). The seismic design of steel structures will be in accordance with the additional provisions of TI 809-04, "Seismic Design for Buildings". Guidance on cold-formed steel can be found in TI 809-07, "Design of Load Bearing Cold-Formed Steel Systems and Masonry Veneer / Steel Stud Walls".

5.3.2 COLD-FORMED STEEL. The design of cold-formed carbon and low alloy steel structures will be in accordance with the TI 809-07. The design of cold-formed stainless steel members will be in accordance with ASCE 8, "Specifications for the Design of Cold-Formed Stainless Steel Structural members. Composite slabs of concrete on steel deck will be designed and constructed in accordance with ANSI/ASCE 9, "Standard for the Structural Design of Composite Slabs". The seismic design of cold-formed steel structures will be in accordance with the additional provisions of TI 809-04.

5.3.3 STEEL JOISTS. The design, manufacture, and use of open web steel joists and joist girders will be in accordance with one of the following Steel Joist Institute (SJI) specifications:

- Standard Specifications for Open Web Steel Joists, K Series
- Standard Specifications for Longspan Steel Joists, LH Series, and Deep Longspan Steel Joists, DLH Series, or
- Standard Specifications for Joist Girders

5.3.4 STEEL CABLES STRUCTURES. The design, fabrication, and erection, including related connections, and protective coatings of steel cables for buildings will be in accordance with ASCE 19, "Structural Applications of Steel Cables for Buildings" except as modified as follows for seismic design:

- Section 5d of ASCE 19 will be modified by substituting (1.5 T4) where T4 is the net tension in the cable due to dead load, prestress, live load, and seismic load. A load factor of 1.1 will be applied to the prestress force to be added to the load combination of Section 3.1.2 of ASCE 19.

5.3.5 CRANE RUNWAYS AND SUPPORTS.

5.3.5.1 STOPS AND BUMPERS. 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 resilient or other energy absorbing construction designed to limit the deceleration force resulting from the crane's hitting the runway stops. Stops engaging the thread of the wheel 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. 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.

5.3.5.2 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.

5.4 SERVICEABILITY.

5.4.1 GENERAL. AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD) provides design guidance with respect to the following serviceability related issues:

- Camber
- Expansion and contraction
- Deflection, vibration and drift
- Connection slip
- Corrosion

5.4.2 CAMBER. Camber is especially important in truss type structures such as those typically used over rolling doors in aircraft hangars. In general, however, the use of camber to offset dead load deflections in long span beams, girders, and joists has limited benefits with respect to building serviceability. Camber however can improve the appearance of structures where the support systems are exposed to view.

5.4.3 EXPANSION AND CONTRACTION. Expansion joints should be provided when necessary to minimize detrimental effects resulting from the lateral movement of long structures due to thermal expansion and contraction. In general, for non-rectangular steel-framed buildings, the maximum allowable building length without an expansion joint (or maximum length between expansion joints) is 90 meters (300 feet) for moderate climates (temperature change less than 17^{o} C (30^{o} F)). This decreases to 60 meters (200 feet) for extreme climates (temperature change greater than 20^{o} C (36 ^{o} F)). These distances may be increased for rectangular buildings, however, they should be decreased if the building is unheated, if the building is air conditioned, or if the building has fixed-base columns. Additional information on expansion joints in buildings can be found in Technical Report No. 65, "Expansion Joints in Buildings," Academy of Sciences, 1974. A double column arrangement is the preferred method of establishing an expansion joint. Additional joints may be required at the junctures of T-, L-, U-shaped and other irregularly shaped buildings, or when foundation conditions create a potential for differential settlement.

5.4.4 DEFLECTION, VIBRATION AND DRIFT. Displacements of structural framing system under service load conditions must be controlled to prevent damage to various architectural features such as interior walls, partitions, ceilings, and exterior cladding. Deflection limits, expressed as a function of span length are provided in various codes (AISC, AISI, MBMA, etc.). Drift limits for earthquake loadings are provided in FEMA 302. Designers should verify the deflection and drift limits imposed by code are suitable. Drift limits more stringent than those imposed by FEMA 302 may be warranted for those conditions where non-ductile cladding is used.

5.4.5 CONNECTION SLIP. When connection slip can cause a significant displacement increase in the framing system and thereby raise concerns about building serviceability, the use of slip critical connections should be considered.

5.4.6 CORROSION. Corrosion can reduce structural capacity as well as cause serviceability problems. Painting of structural steel will comply with the requirements contained in either AISC-LRFD or AISC-ASD. Except where fabricated of approved corrosion resistant steel or of steel having a corrosion resistant or other approved coating, individual structural members and assembled panels of cold-formed steel construction must be protected against corrosion with an approved coat of paint.

5.4.7 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.

5.5 LOAD PATH INTEGRITY.

5.5.1 GENERAL. Connections between framing elements are critical. They must perform at limit state and service load levels as intended to assure load path integrity is maintained during extreme loading events. Under service loading conditions, connection displacements and rotations should not lead to serviceability problems. Connections can be welded or bolted, or a combination of both welds and bolts. Design of welded and bolted connections will be in accordance with either the AISC Load and Resistance Factor Design (AISC-LRFD) or AISC Specification for Structural Steel Buildings-Allowable Stress Design (AISC-ASD). Bolted connections can be shear-bearing type connections, slip critical connections, or direct tension connections. Field welding of connections should be minimized to the maximum extent possible.

5.5.2 DESIGN OF CONNECTIONS FOR WIND AND SEISMIC LOADS. Connection design philosophy is different for earthquake than it is for wind. Connections used to resist wind load are designed to perform in the elastic range. Connections used to resist earthquake loads, although designed in a fashion similar to that for wind load, are expected to experience forces greater than the code level design forces. This requires that earthquake resistant connections perform in a ductile manner. Under certain circumstances, such as moment frame connections, bearing connections may be desirable for wind load, but slip critical connections may be recommended to resist earthquake loads. Slip critical connections are recommended for earthquake resistant beam-column connections where web connections are bolted and flange connections are welded. Slippage of the bolts in these type connections can increase loads on the flange welds resulting in connection failure (Reference UCB/EERC-83/02 Report, "Seismic Moment Resisting Connections for Moment-Resisting Steel Frames": Keep in mind that the ultimate load capacity of a slip critical connection is the capacity of the bolts loaded in shear/bearing. If slip critical bolts in the web connection of a moment resistant joint are too near the yielding flanges, the bolts may experience sufficient force to slip into a shear/bearing type of response, which may have a lower capacity than the slip-critical type of response.). There are significant differences between the strengths allowed for shear-bearing, and slip critical bolts. Therefore, the designer must check both wind and seismic loading conditions to make sure the connection satisfies both shear-bearing and slip critical requirements.

5.5.3 SHEAR-BEARING TYPE CONNECTIONS. In a shear-bearing type connection, shear forces are transferred through the connection by bolts that act in shear. The connected material is in bearing adjacent to the bolts and must be evaluated for its load bearing capacity. Bolt shear strength is about 62 percent of the bolt-ultimate tensile strength. It should be recognized that all shear and moment connections at ultimate load conditions act in shear-bearing whether they are designed as shear-bearing connections or as slip-critical connections. The bolts in shear-bearing type connections are installed "snug tight" since shear strength in this type of connection is not related to the pretension load in the bolt. The bolts are usually ASTM A307 bolts with an ultimate tensile strength of 450 MPa (60ksi), although "snug tight" high-strength ASTM A325 and ASTM A490 bolts are also permitted for shear-bearing type connections with some restrictions. ASTM A325 bolts have an ultimate tensile strength of 830MPa (120 ksi), and ASTM A490 bolts have an ultimate tensile strength of 1040 MPa (150 ksi).

5.5.4 SLIP CRITICAL CONNECTIONS. Slip critical connections transfer shear load by shear-friction with the normal force provided by pretensioning the bolts. The shear strength at service load levels is dependent on the pretension load (normal force on joint) and the friction between joined materials. Slip critical connections are generally required when loads are repetitive and fatigue is a concern, where connection slip would effect structure performance and where bolts and welds must act in unison to resist applied loads. Bolts are pretensioned to at least 70 percent of the bolt-minimum tensile strength. The tension load is applied by tightening the nut, or otherwise elongating the bolt sufficiently to develop the prescribed pretensioning force. Turn-of-the-nut, calibrated wrench, and direct tension indicators are all methods used to assure the bolt has been properly tensioned.

5.5.5 BOLTS IN DIRECT TENSION. When bolts act in tension to resist applied loads, the possibility of, and the effect of prying action on bolt tension must be considered.

5.5.6 STEEL PROPERTIES. Strength and ductility are properties important to building performance. Ductility is dependent on the yield strength and chemical composition of the steel. Buildings located in high seismic areas must be ductile if they are to survive major earthquakes. Designers must select steels that meet needed strength and ductility requirements. They must recognize that ASTM A572 Grade 50 steel will meet ASTM A36 requirements and most likely will be used even though ASTM A36 steel is specified. Designers specifying ASTM A36 steel must consider the impact a higher yield strength will have on the seismic design and make sure the substitution does not adversely impact seismic performance. Connections designed to develop the strength of the connected members may not perform as intended if the connected members were specified to be ASTM A36 steel and the contractor elected to substitute members of ASTM A572, Grade 50 steel.

5.6 SPECIAL INSPECTIONS.

5.6.1 PERIODIC SPECIAL INSPECTION of the installation and tightening of fully tensioned high strength bolts in slip critical connections and in connections subject to direct tension is required.

5.6.2 CONTINUOUS SPECIAL INSPECTION of all structural welding, except where periodic special inspection is allowed in FEMA 302, Chapter 3, is required.

6. METAL DECKS

6.1 INTRODUCTION. This section prescribes the criteria and procedures for the design of metal roofing and steel deck diaphragms for buildings.

6.2 METAL ROOFING. Metal roofing consists of cold-formed, corrugated, fluted or ribbed metal sheets attached to the exterior of building structures and exposed to weather to serve as the exterior covering of the structure. Metal roofing may be either structural, or non-structural. The structural metal roof is designed so the roofing panels support all out of plane loads. Forces in the plane of the roof due to lateral wind and earthquake forces are typically resisted by steel deck diaphragms, or in plane X-bracing. The non-structural roof depends on substrate to carry the applied loads. Metal roofing may have lapped side seams and exposed fasteners, standing side seams and hidden metal clip fasteners, or hybrid types such as those with snap seams or battens that fall somewhere in between. There are many types of metal roofing produced by the metal manufacturing industry and care must be exercised to ensure that the type specified is compatible with the main structural system or substrate.

6.3 METAL DECK DIAPHRAGMS.

6.3.1 METAL DECK DIAPHRAGMS without Structural Concrete Topping. Metal deck diaphragms without structural concrete topping are usually used for roofs of buildings where the gravity loads are light (live loads are 1000 kg/m^2 (20 psf or less). The metal deck units are often composed of steel sheets ranging in thickness from 0.75 mm (0.03 inches) to 2 mm (0.08 inches). The sheets are 600 mm (2 feet) to 900 mm (3 feet) wide, and formed in a repeating pattern with ridges and valleys. Rib depths vary from 40 mm to 100 mm (1-1/2 to 4 inches) in most cases. Decking units are attached to each other and to the structural steel supports by welds or mechanical fasteners. Chords and collector elements in these diaphragms are composed of steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collectors is thorough shear connectors, puddle welds, screws, or shot pins.

6.3.2 METAL DECK DIAPHRAGMS WITH STRUCTURAL CONCRETE TOPPING. Metal deck diaphragms with structural concrete topping are frequently used on floor and roofs of buildings where the loads are moderate to heavy. The metal deck may be either a composite deck, which has indentations, or a non-composite deck. In both cases the slab and deck act together to resist diaphragm loads. The concrete fill may be normal weight or lightweight concrete, with reinforcing composed of wire mesh or small diameter reinforcing steel. Additional reinforcing may be added in areas of high stress. The metal deck units are composed of gage thickness steel sheets, 600 mm (2 feet) to 900 mm (3 feet) wide, and are formed in a repeating pattern with ridges and valleys. Decking units are attached to structural steel supports by welds or mechanical fasteners. The concrete topping has structural properties that significantly add to diaphragm stiffness and strength. The topping should be a minimum of 65 mm (2-1/2 inches) thick. Concrete reinforcing ranges from light mesh reinforcement to a regular grid of small reinforcing bars. Metal decking is typically composed of corrugated sheet steel from 22 gage down to 14 gage. Rib depths vary from 40 mm to 75 mm (1-1/2 inches to 3 inches) in most cases. Chord and collector elements in these diaphragms are considered to be composed of the steel frame elements attached to the diaphragm. Load transfer to frame elements that act as chords or collector elements is usually through welds or headed studs.

6.3.3 METAL DECK DIAPHRAGMS WITH NONSTRUCTURAL CONCRETE TOPPING. Metal deck diaphragms with nonstructural concrete fill are typically used on roofs where gravity loads are light. The concrete fill, such as very lightweight insulating concrete (e.g. vermiculite), does not have useable structural properties. If the concrete is reinforced, reinforcing steel consists of wire mesh or small diameter reinforcing steel. To act as a diaphragm load transfer must be the same as that provided for a metal deck diaphragm without concrete topping.

6.3.4 HORIZONTAL STEEL BRACING (STEEL TRUSS DIAPHRAGMS). Horizontal steel bracing (steel truss diaphragms) is often used in conjunction with structural metal roofing systems where the strength and stiffness of the metal roofing is incapable of carrying in-plane loads. Steel truss diaphragm elements are typically found in conjunction with vertical framing systems that are of structural steel framing. Steel trusses are more common in long span situations, such as special roof structures for arenas, exposition halls, auditoriums, industrial buildings, and aircraft hangars. For steel truss elements with large in-plane loads, diagonal elements may consist of angles, tubes, or wide flange shapes that can act in both tension and compression. Diagonals which can act in both tension and compression are preferred, however with lightweight metal buildings the diagonals are often steel rods which can act only in tension. Sufficient load path reliability should be provided for diagonals in accordance with the redundancy recommendations in the technical literature. Truss element connections are generally concentric, to provide the maximum lateral stiffness and ensure that the truss members act under pure axial load.

6.4 BASIS FOR DESIGN.

6.4.1 METAL ROOFING. The basis for the design of metal roofing is provided in TI 809-29, "Structural Considerations for Metal Roofing."

6.4.2 METAL (STEEL) DECK DIAPHRAGMS. Steel deck diaphragms will be made from materials conforming to the requirements of the American Iron and Steel Institute (AISI), Specifications for the Design of Cold-formed Structural Members," or ANSI/ASCE 8, "Specifications for the Design of Cold-formed Stainless Structural Steel Members."

6.4.2.1 IN-PLANE LOADS. Nominal in-plane shear strengths will be determined in accordance with approved analytical procedures. Design strengths will be determined by multiplying the nominal strength by a resistance factor, ϕ, equal to 0.60 for mechanically connected diaphragms, and equal to 0.50 for welded diaphragms. Analytical procedures contained in the Steel Deck Institute, Inc., "Diaphragm Design Manual #DDM01" are accepted means for calculating in-plane shear strengths. Limits are placed on diaphragm span and depth to span ratios to keep diaphragm in-plane displacements small enough to prevent cracking of walls. The maximum span and span to depth ratio depends on diaphragm stiffness and wall ductility. These and other additional requirements for diaphragms and their connections are provided in TI 809-04, "Seismic Design for Buildings."

6.4.2.2 OUT-OF-PLANE LOADS. Design loads and design requirements for out-of-plane loads for bare metal deck roofing will be in accordance with TI 809-29, "Structural Considerations for Metal Roofing." The design for out-of-plane loads for metal deck with structural concrete topping (composite steel floor deck) will be in accordance with the Steel Deck Institute, Inc., "Design Manual for Composite Decks, Form Decks, and Roof Decks."

6.5 METAL DECK DIAPHRAGMS - STIFFNESS FOR ANALYSIS (FEMA 273). Diaphragms can be considered to be flexible, or rigid. For flexible diaphragms, the lateral forces are distributed from the metal deck diaphragm to the vertical lateral force resisting elements by assuming the diaphragm acts as a simple beam spanning between vertical lateral-force resisting elements. For rigid diaphragms, the lateral forces are distributed to the vertical lateral-force resisting elements based on the relative stiffnesses of the vertical lateral-force resisting elements. Flexibility factors, provided in manufacturers' catalogs as well as in the Diaphragm Design Manual of the Steel Deck Institute, can be used to determine whether the diaphragm should be considered as flexible or rigid. For bare metal deck diaphragms the stiffness is a function of metal thickness, rib geometry, fastener type, and fastener spacing. Procedures for calculating diaphragm flexibility are also provided in TI 809-04, "Seismic Design for Buildings."

6.6 SPECIAL INSPECTIONS.

- Periodic special inspection is required for all welding of all steel deck and steel truss elements of the seismic force resisting system.

- Periodic special inspections are required for screw attachment, bolting, anchoring, and other fastening components within the seismic force resisting system including diaphragms, drag struts, collector elements, and truss elements.

7. WELDING

7.1 INTRODUCTION. A weldment may contain metals of different compositions, and the components of the framing system may be rolled shapes, pipes, tubes, or plates. In general, all metals are weldable, but some are much more difficult to weld than others. Certain types of reinforcing steels used in concrete and masonry construction can be very difficult to weld. Various types of joints are used to connect framing components, including butt joints, corner joints, edge joints, lap joints, and T-joints. Various types of welds are used in joining steel components together, and several different welding processes can be used to make the weld. Structural engineers involved in the design of weldments must be familiar not only with the various types of joints and types of welding procedures, but also with the effect welding has on the selection of welding procedures, and with the effect factors such as base metal composition, electrode selection, preheat and interpass temperature, wire feed speed, travel speed, post weld treatment, ambient temperature, etc., have on weld performance. Volume change effects due to temperature gradients that occur due to welding, joint restraint conditions, and weld toughness are factors which must also be considered to assure weldments will be free from cracks that could reduce joint capacity and ductility. Corner joints and poor weld termination can cause stress concentrations that can also lead to weldment cracking.

7.2 BASIS FOR DESIGN. Weldments will comply with the American Welding Society (AWS) AWS D1.1, "Structural Welding Code - Steel". AWS D1.1 contains many prequalified joint details that are known to produce quality weldments when fabricated in accordance with AWS Welding Procedure Specifications (WPS). WPS's are required for all welding, including prequalified procedures. Additional guidance on welding can be found in TI 809-26, "Welding Guidance for Buildings."

7.3 ARC WELDING PROCESSES. The most popular arc welding procedures are Shielded Metal Arc Welding (SMAW) and Flux Core Arc Welding (FCAW). The SMAW process is defined as "an arc welding process that produces coalescence of metals by heating them with an arc between a covered metal electrode and the work. Shielding is obtained from decomposition of the electrode covering. Pressure is not used and filler metal is obtained from the electrode." The FCAW process is defined as "an arc welding process that produces coalescence of metals by heating them with an arc between a continuous filler metal (consumable) electrode and the work. Shielding is produced by a flux contained within the tubular electrode. Additional shielding may or may not be obtained from an externally supplied gas or gas mixture". Shielding refers to methods used to prevent the molten weld metal from coming in contact with gasses contained in the surrounding air. These gases, especially oxygen, nitrogen, and hydrogen, are the most detrimental to weld quality. Other types of arc welding processes are carbon arc welding (CAW), gas tungsten arc welding (GTAW), plasma arc welding (PAW), submerged arc welding (SAW), and gas metal arc welding (GMAW). The type of base metal usually determines which welding processes can be used.

7.4 WELDING POSITIONS. Welding positions include the flat position, the horizontal position, the vertical position, and the overhead position. Certain welding processes have limitations on which welding positions can be used. The structural engineer designing the weldment must be familiar with these restrictions, and make sure the type of weldment specified can actually be applied as intended considering access conditions, weld position, and any environmental conditions that could adversely influence weldment fabrication.

7.5 WELDMENTS SUBJECTED TO EARTHQUAKE AND CYCLIC LOADING CONDITIONS. Certain types of welds are not permitted to join members that may experience cyclic loadings due to earthquake ground motions, or due to vibratory motion caused by equipment. Partial penetration butt joints in tension, intermittent groove welds, and intermittent fillet weld are examples. Special Moment Resisting Frames (SMRF's) and Ordinary Moment Resisting Frames (OMRF's) used for earthquake resistance must be capable of performing in the nonlinear range (beyond yield) during major earthquakes. For SMRF's this means the weldments designed for beam column joints must have greater strength than the connected members so that yielding occurs in the beam span away from the beam column joint. Welded steel frame design for seismic resisting moment frame systems is covered by the American Institute of Steel Construction (AISC), Seismic Provisions for Structural Steel Buildings".

7.6 WELDMENT STRENGTH AND DUCTILITY. Base metals selected for components of the structural framing system must possess certain strength and ductility characteristics. It is important that the electrodes for joint weldments, as well as welding procedures, be selected to produce a joint with strength and ductility properties equal to, or superior to, those of the base metal.

7.7 ENVIRONMENTAL FACTORS IMPORTANT TO WELDMENT PERFORMANCE. Cracking can occur due to environmental factors such as the presence of moisture, and low ambient temperatures. Field welding should be avoided as much as possible. However, when field welding is required, especially under adverse environmental conditions, it is important that AWS welding specification procedures be followed to the letter. Preheating and inter-pass temperatures are critical if cracking is to be prevented. Preheating also helps to drive off excess surface moisture, and retards the cooling rate thereby minimizing temperature gradients. Controlling cool-down rates, especially during cold weather, is critical if cracking is to be eliminated.

7.8 WELDING REINFORCING STEEL. The welding of reinforcing steels is covered by AWS D 1.4, "Structural Welding Code - Reinforcing Steel." Most reinforcing steel bars can be welded. However, the preheat and other quality control measures that are required for bars with high carbon equivalents are extremely difficult to achieve. It is recommended that carbon equivalents be limited to 0.45 percent for No. 23 bars (No. 7 bars) and higher, and to 0.55 percent for smaller bars. ASTM A615, Grade 60 reinforcing steel will most likely not meet the aforementioned carbon equivalent requirements. However, reinforcing bars meeting ASTM Specification A706 have a low carbon equivalent, are easy to weld, and should be considered when welding is required. Mechanical connectors are another way of connecting reinforcing bars.

7.9 SPECIAL INSPECTIONS

- Continuous special inspection for all structural welding, except periodic special inspection is permitted for single pass fillet or resistance welds and welds loaded to less than 50 percent of their design strength provided the qualifications of the welder and the welding electrodes are inspected at the beginning of the work and all welds are inspected for compliance with the approved construction documents at the completion of welding.

- Continuous special inspection is required during the welding of reinforcing steel used in concrete or masonry construction.

8. WOOD

8.1 INTRODUCTION. This section provides a list of guidance documents to be used 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 included either in this chapter or in the referenced guidance documents in Appendix A. Guidance documents referenced include design standards and specifications. The use of timber construction will consider the type of occupancy and meet all fire protection criteria and requirements. Detailed design information on wood buildings is not provided because wood construction is generally limited to residential construction since strict fire protection standards preclude the use of wood construction for most other types of buildings.

8.2 BASIS FOR DESIGN. The design of structural elements or systems constructed partially or wholly of wood or wood-based products will be by allowable stress design or load and resistance factor design. The structural analysis and construction of wood elements and structures using allowable stress design methods will be in accordance with the applicable standards. The structural analysis and construction of wood elements and structures using load and resistance factor design methods will be in accordance with AF&PA/ASCE 16, "Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction." The design and construction of wood structures to resist seismic forces and the material used therein will comply with the requirements of TI 809-04, "Seismic Design for Buildings," and FEMA 302, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures."

8.3 SERVICEABILITY CONSIDERATIONS.

8.3.1 CLIMATIC CONSIDERATIONS. Engineering properties usually are not appreciably affected when wood is subjected to extremely low temperatures. For cold region limitations on wood construction, see TM 5-852-9/AFR 88-19, Volume IX. The engineering properties of wood are not appreciably affected in tropical climates. Rot and insect attacks, however, are aggravated tropical humid areas, and all timber for permanent construction in tropical areas should be preservative treated. Bonding of wood to wood can be made by a variety of adhesives. In tropical climates, structural bonding of wood to other materials should be by means of epoxy resin adhesive.

8.3.2 FIRE RETARDANT TREATMENT. Recommendations regarding the use of fire retardant treatments are provided in the USDA Wood Handbook and the National Fire Protection Handbook. Pressure impregnation is the preferred treatment method.

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

8.3.4 ORIENTED STRAND BOARD. The use of oriented strand board (OSB) for non-vertical applications is not permitted. For floor and roof sheathing, APA structural rated plywood sheathing only will be used. Specifically, for floors, use as a minimum, 18mm (23/32 inch) thickness APA rated STURD-I-FLOOR, 600 mm (24inches) on center span rating, Exposure 1, Tongue and Grove, glued and nailed. In addition, all of the requirements of the APA "Code Plus Floor" will be met. Ring or screw-shank nails will be used.

8.4 SPECIAL INSPECTIONS. Continuous special inspection during all field gluing of elements of the lateral force resisting system is required. Periodic special inspections for nailing, bolting, anchoring, and other fastening of components within the lateral forces resisting system, including drag struts, braces, and tie-downs, is required.