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Array Structure Design Handbook for Stand Alone Photovoltaic Applications

Robert C. Didelot
National Aeronautics and Space Administration
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Prepared for
U.S. DEPARTMENT OF ENERGY
Conservation and Renewable Energy
Division of Photovoltaic Energy Systems

NOTICE

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FOREWORD

The preliminary draft of this handbook was prepared by InterTechnology/Solar Corporation for the NASA Lewis Research Center under Contract DEN 3-33. This draft was subsequently reorganized, revised, and expanded at the Center. In addition, an evaluation envolving expansion and critical analysis of the tables and other portions of the handbook was performed by Cleveland State University under NASA Cooperative Agreement NCC 3-16. G. C. Chang and M. H. Ellini of Cleveland State University participated in the verification, review and analysis check of critical portions of the handbook. The final editing was then conducted by NASA Lewis Research Center personnel.

TABLE OF CONTENTS

1.0 INTRODUCTION

This *Array Structure Design Handbook for Stand Alone Photovoltaic Applications* is a self-contained reference manual written for nonstructural engineers. The handbook is written in the format of an engineering guide and contains information sufficient to enable engineers unfamiliar with photovoltaic systems to design low-cost array structures for a variety of terrestrial photovoltaic applications.

The scope of the handbook is described below:

A. Array Size

The handbook is written to facilitate design of photovoltaic arrays of up to 10 kilowatt peak output power (kWp). The design principles incorporated in this handbook, however, may be used to design and construct dedicated triangular frame arrays larger than 10 kWp, if so desired.

B. Structural Types

The handbook covers dedicated and portable triangular framing and pole-mounted systems. (See Figures 11, 12, and 13.)

C. Structural Materials

The major structural materials covered in this handbook are: structural steel (A-36), aluminum (6060-T6), and cold-formed steel. Users should consult Section 4.6.

D. Structural Section Sizes

All designs discussed in this handbook are for standard U.S. section sizes. The dimensions of the standard American sections are given in English units only.

E. Location of Application

The directions given in this handbook can be applied anywhere in the world. For applications outside the United States, however, additional data may have to be secured from other sources, particularly for site specific wind loads, frost penetration data, and local building codes.

F. Module Type

The instructions given in this handbook can be applied to any commercially available solar cell module.

G. Units Used

The primary system of units used throughout the handbook is the International System of Units (SI Units). For the most part, English Units were used initially in developing the material for the handbook, then subsequently converted to SI Units for presentation in the handbook. In order to enhance communication and utility of the handbook, English Units are also given either parenthetically following the SI Units (text and figures) or as separate columns (in the tables). An exception was made, however, for some of the tables and figures and the references which were extracted directly from reference sources. In these cases, the units utilized in the source publications are retained.

H. Terminology Used

In order to clarify terminology used throughout the handbook which is unique to photovoltaic arrays the user should become familiar with several important terms and definitions. These are as follows:

Solar Cell: The basic photovoltaic device which generates electricity when exposed to sunlight.

Module: The smallest complete, environmentally protected assembly of solar cells and other components (including electrical connectors) designed to generate d.c. power when under unconcentrated terrestrial sunlight.

Array: A mechanically integrated assembly of modules together with support structure (including foundations) and other components, as required, to form a free-standing field-installed unit that produces dc power.

I. Handbook Format

This handbook is divided into four primary sections as follows:

2.0 KEY TO THE MANUAL
3.0 CHARACTERISTICS OF PHOTOVOLTAIC ARRAY STRUCTURES
4.0 ARRAY STRUCTURE DESIGN
5.0 ARRAY/STRUCTURAL DESIGN EXAMPLE PROBLEMS

The KEY TO THE MANUAL (Section 2.0) gives the engineer a step-by-step methodology on how to use the handbook to design a complete structural array system.

Section 3.0 discusses the characteristics of array structures, including module characteristics, array structural characteristics, electrical and mechanical interfaces, safety considerations, and array layout and structural arrangements.

Section 4.0 explains the design factors for photovoltaic array systems, including: general design philosophy; building codes; loading and environmental factors; types of generic structures covered; the basic design methodology used; materials selection; design of triangular framing and pole-mounted structural systems, including design procedures, material properties and specifications, design figures, and design tables; installation and maintenance; and costing.

Section 5.0 includes several array/structural design examples making use of the material presented in Sections 2.0, 3.0, and 4.0.

Also included in the handbook are a GLOSSARY (Section 6.0) which includes a list of definitions and conversions; REFERENCES (Section 7.0) which include additional design sources, lists of manufacturers, and extracts from auxiliary design references; and APPENDICES (Section 8.0) which contain other relevant auxiliary information.

2.0 KEY TO THE MANUAL

This handbook will prove to be an easy-to-use reference source for the architect or engineer who follows the step-by-step procedure given below for designing a structural system for photovoltaic arrays. The procedure consists of eleven basic steps illustrated in the flow diagram in Figure 1 and as described in detail below. The order of the steps listed below may vary slightly for applications outside the scope of the handbook. As one example, when generic structural systems in addition to the triangular frame or pole-mounted system are being considered, Step 3 would generally be finalized after Steps 4 and 5.

A. Step 1--Gather Preliminary Data

(a) With reference to the array site, compile the following information:

(1) Latitude
(2) Longitude
(3) Yearly weather conditions, particularly wind, rain and humidity, snow, hail, ice, temperature ranges and pollution levels.
(4) Size, shape and topography of field on which the array is to be placed
(5) Shading from nearby structures or trees
(6) Location of existing underground electrical and water lines
(7) Soil sample (optional)

(b) Determine the array size. It is outside the scope of this handbook to discuss photovoltaic system design or array sizing techniques, therefore, other sources will need to be consulted in order to specify the specific voltage, current and power requirements and optimum tilt angle for the array.

(c) Consult Section 4.1 for the philosophy of structural design.

B. Step 2--Consult the Zoning and Building Codes

Turn to Section 4.2 for reference on zoning and building codes. There are no codes written expressly for photovoltaic arrays, but codes for other structures may be applied.

C. Step 3--Select a Generic Structure

Go to Section 3.3 for the basic support requirements for the module. Then, turn to Section 4.5 to select a generic structure which could be either a dedicated or portable triangular framing system, or a pole-mounted system. Section 4.5 contains the basic design theory for triangular frame structures.

D. Step 4--Choose the Module Type

Choose a module type and determine the number of modules and their arrangement in series and parallel groupings. Turn to Sections 3.1 and 3.2

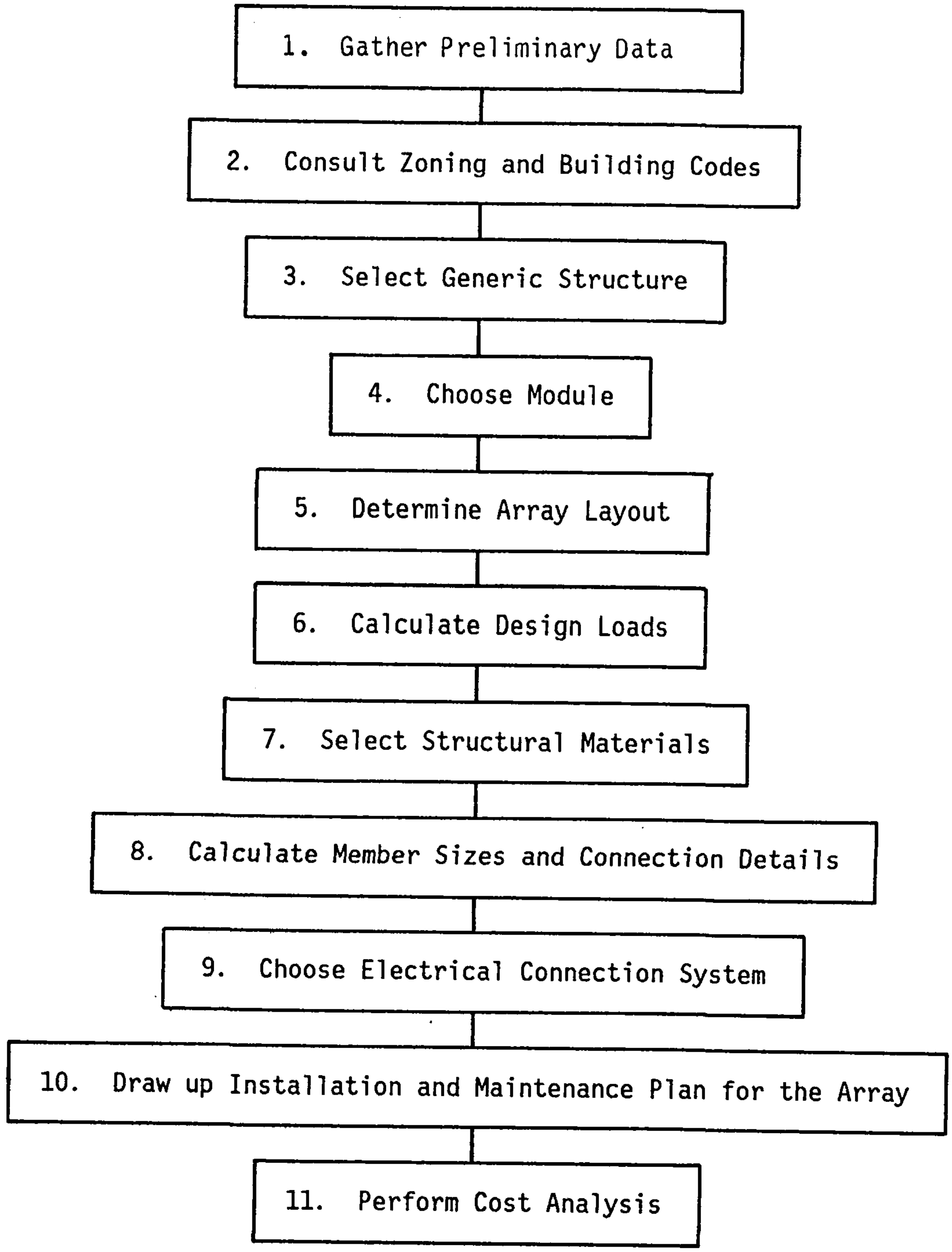


Figure 1. Design Procedure Flow Diagram

for a discussion on module mechanical and electrical and thermal characteristics. Section 7.3.4 lists current manufacturers of modules in the United States. Sections 3.1.3 and 4.4 should be consulted to make sure that the module chosen for the array is constructed of suitable materials to survive the environment without extensive degradation.

E. Step 5--Determine the Array Layout

After the module type has been chosen determine the dimensions of the array and decide on the number of tiers, span width, and number of rows of structure required. Turn to Section 3.5 for a discussion of the array layout and structural arrangements.

F. Step 6--Calculate the Design Loads for the Array Structure

Turn to Section 4.3 to determine the load factors pertinent to array structures.

G. Step 7--Select Suitable Construction Material

Turn to Sections 4.4 and 4.6 for discussions on environmental effects on structural materials and descriptions and characteristics of structural steel, aluminum, cold-formed steel, and wood. Then turn to Sections 4.7.1.2, 4.7.1.3, 4.7.1.4, and 4.7.1.5 which cover design, fabrication, and construction specifications for structural steel, aluminum, cold-formed steel, and foundations.

H. Step 8--Calculate Member Sizes and Connection Details

(a) Turn to Section 3.3 to determine the mechanical support requirements for the array.

(b) Turn to Section 3.4 to determine the alternative mechanical connection systems for connecting the module to the array structure.

(c) Turn to Section 4.7.1.1 and follow the procedure listed to determine member sizes, foundation, and connection details for triangular framing systems.

(d) Turn to Section 4.7.2.1 and follow the procedure listed to determine the size of pole, foundation, member sizes, and connection details for pole-mounted systems.

I. Step 9--Choose an Electrical Connection System

Refer to Section 3.4 to choose a basic electrical connection system. It is outside the scope of this handbook to discuss electrical connection details such as wire or duct size. The basic system, however, may be chosen after reading the section cited.

J. Step 10--Draw up Installation and Maintenance Plan for the Array

Section 4.8 presents the methodology for writing an installation and maintenance plan, including procurement of all material, necessary equipment, and plans for maintenance.

K. Step 11--Perform a Cost Analysis

Section 4.9 presents the principles for performing a cost analysis, including primary cost factors and sources of information.

3.0 CHARACTERISTICS OF PHOTOVOLTAIC ARRAY STRUCTURES

The characteristics of photovoltaic arrays which are of interest to the designer of an array structure are the module mechanical and electrical characteristics, module thermal characteristics, module/array structural support requirements, module installation and removal factors, and the array layout and structural arrangement.

3.1 Module Mechanical and Electrical Characteristics

A photovoltaic module is a rectangular system consisting of solar cells, a substrate on which the cells are mounted, a superstrate covering the cells for protection, and a frame in which the cells, substrate and superstrate are mounted. Figure 2 illustrates a typical photovoltaic solar cell module. Currently there are about twenty companies around the world which manufacture photovoltaic modules.

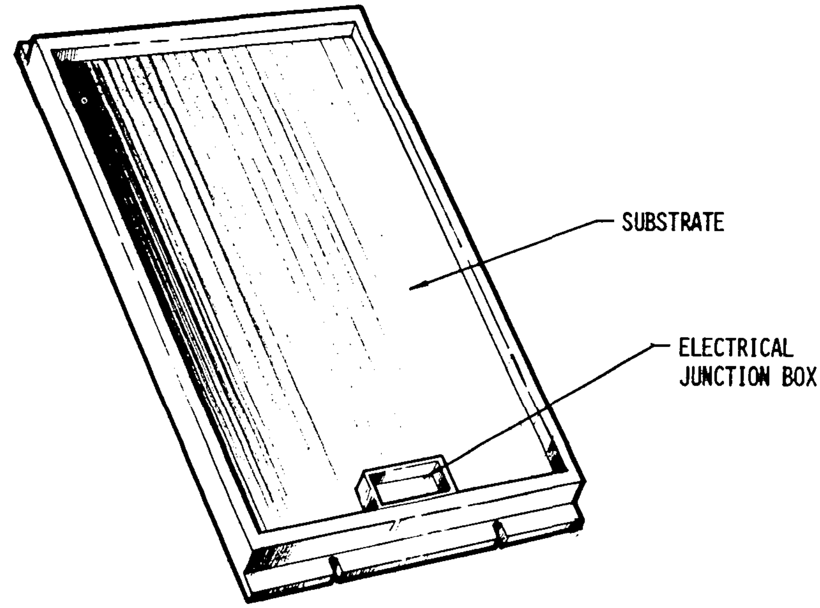
A sampling of U.S. manufacturers of photovoltaic modules is given in Section 7.3.4. From the standpoint of array structural design, there are six major module characteristics to be considered: efficiency and output, size, materials, weight, provisions for mechanical connections and provisions for electrical connections.

3.1.1 Efficiency and Output

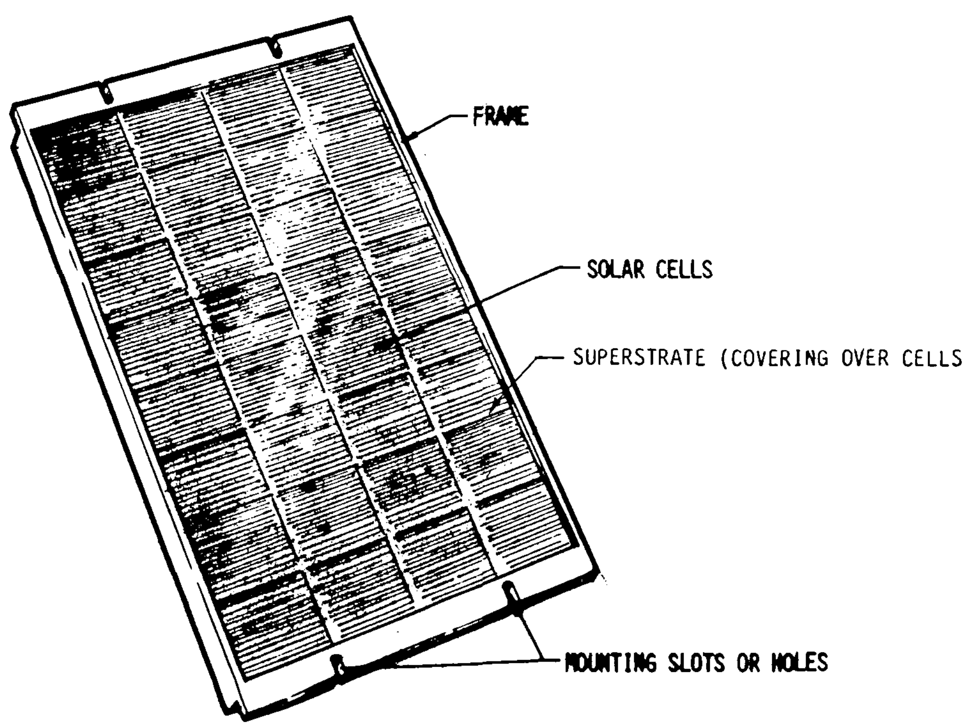
The module efficiency may be defined as the output power of the module divided by the insolation level (input power), where the maximum terrestrial insolation level is 1 kW/m^2. Module efficiency may range from a low of about 6 percent to a high of about 16 percent. These efficiencies correspond to peak power densities of 60 Wp/m^2 to 160 Wp/m^2, respectively. Most currently manufactured modules have an efficiency and peak power density within the lower two-thirds of these ranges.

Many of the modules presently on the market are sized to trickle charge 12 V batteries. Typically, 14 V to 16 V charging voltage and 0.3 A to 0.6 A charging current are required for battery trickle charging. Since the module current output (as opposed to voltage output) is a direct function of the insolation, the module must have a peak current higher than these values so that it will produce an average current equal to these values. Therefore, correct module (or array) size depends, in large measure, on the amount of insolation available at the site of application.

Modules are also manufactured with different output voltage and current combinations for the same module output power depending upon how the cells in the module are interconnected (i.e., series and parallel connections). For example, one manufacturer offers a 26 Wp 48 cell module in the following types: 21.3 Vm, 5.32 Vm, 3.55 Vm, 2.66 Vm, and 1.77 Vm, where Vm is the voltage at maximum power.



BACK VIEW



FRONT VIEW

Figure 2. Typical Module

3.1.2 Size

Modules currently manufactured vary in size from 0.30 m by 0.15 m (1.0 ft by 0.5 ft) to 1.2 m by 1.2 m (4 ft by 4 ft). Common sizes are 0.30 m by 0.30 m (1 ft by 1 ft), 0.30 m by 0.61 m (1 ft by 2 ft), and 0.61 m by 0.61 m (2 ft by 2 ft), 0.23 m by 1.2 m (0.75 ft by 4 ft), and 0.38 m by 1.2 m (1.25 ft by 4 ft). For a module efficiency of 10 percent, these sizes correspond to module peak output powers of 9.1 W, 18.3 W, 37.2 W, 27.6 W, and 45.6 W, respectively. It should also be noted that some manufacturers may custom-manufacture modules of practically any size.

3.1.3 Materials

Table I shows the types of materials commonly used for the cells, substrate, superstrate and frame. Particular attention should be paid to the substrate material when selecting a module since degradation of the superstrate by the environment may lead to drastically reduced cell performance. Section 4.4 discusses environmental factors associated with this problem.

3.1.4 Weight

The typical module weight varies from 15 kg/m^2 to 40 kg/m^2 (3 lb/ft^2 to 8 lb/ft^2), i.e., 150 N/m^2 to 400 N/m^2, depending on the module construction materials.

3.1.5 Provisions for Mechanical Connections

The most popular provision for mechanical connection of the module to the array structure is a flange with four holes or slots for bolt connections (Figure 2). The bolt connection is usually the simplest and most inexpensive installation method.

Other provisions include flanges or narrow edges without holes for use in clip or clamp mounting.

3.1.6 Provisions for Electrical Connections

Modules differ significantly from one manufacturer to another and with manufacturer types in terms of provisions for electrical connections, as shown in Figure 3. These provisions may be divided into three groups: terminals (Figure 3B), connectors (Figure 3E), and pigtails (Figure 3C).

A. Terminals

The module has two metal terminals to which the positive and negative connecting wires are attached by soldering or by a screw and/or nut connection.

B. Connectors

The module has a connector to which a mating connector is attached for the output.

TABLE I. MODULE CONSTRUCTION MATERIALS

Section	Materials
Substrate	Glass Fiberglass Polyester Stainless Steel Aluminum
Superstrate	Glass Silicone Rubber Silicone Resins Plastics
Frame*	Lexan Stainless Steel Aluminum
Cells	Silicon (Si) [Most Common] Gallium Arsenide (GaAs) Cadmium Sulfide (CdS)

* If substrate does not serve as frame

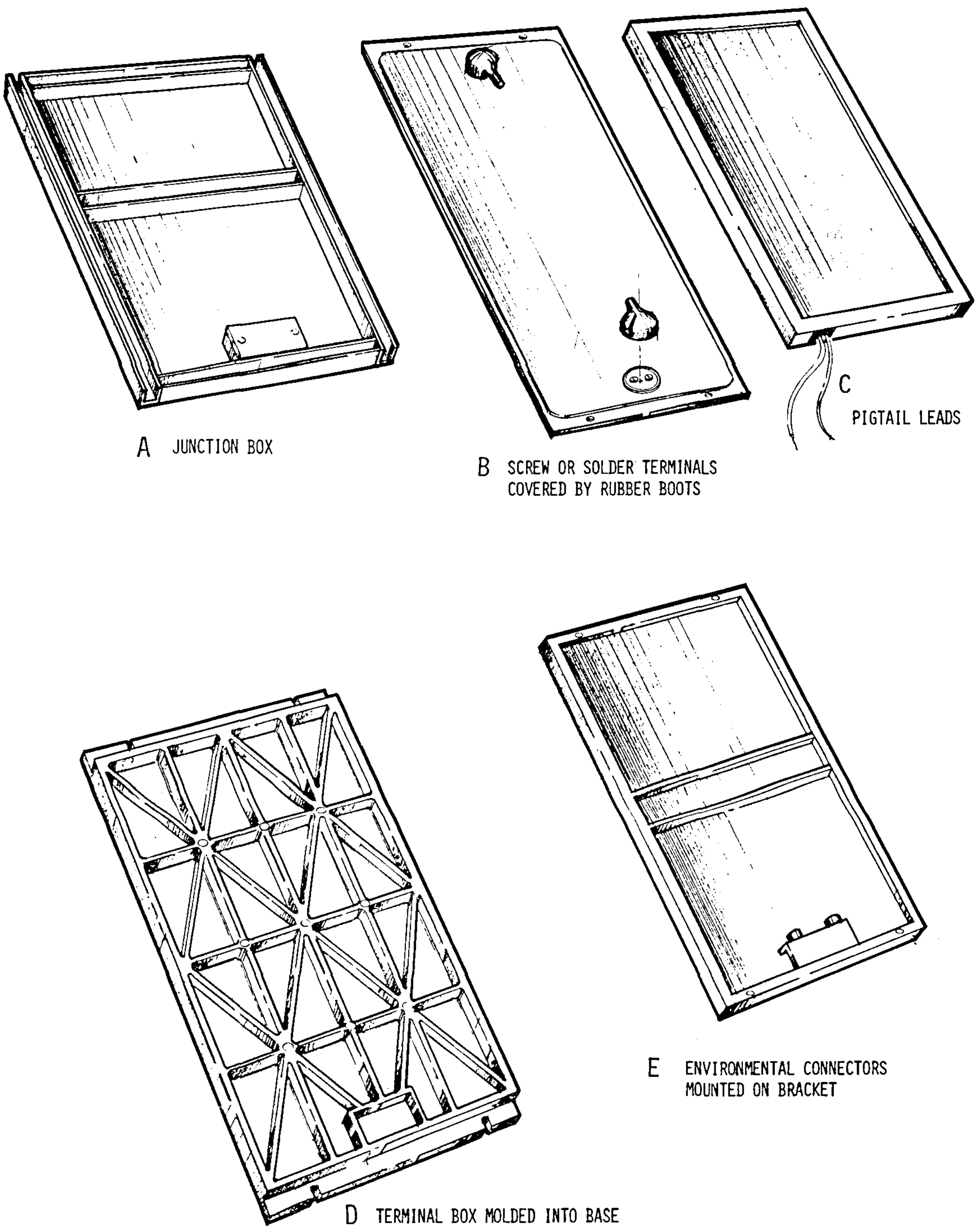


Figure 3. Module Electrical Connections

C. Pigtails

The module has a positive and a negative output lead.

There are various schemes used to protect the electrical connections from the environment. The most common are the junction box (Figures 3A and 3D) and the rubber boot (Figure 3B). Figure 3A illustrates a metal junction box attached to the frame while Figure 3D shows a plastic junction box molded into the base of the module. In extreme environments the junction box should be chosen over the rubber boot as an electrical connection protection.

Any modules with externally exposed metallic conductors require grounding. Grounding methods are discussed in Section 3.4.2.

3.2 Module Thermal Properties

The two major problems involving module thermal properties are thermal expansion and module operating temperature.

3.2.1 Module Operating Temperature

Since flat-plate photovoltaic modules are not actively cooled (i.e., with forced water or air flow), the cell temperature may rise under normal operating conditions to a temperature considerably higher than the ambient temperature. Depending on the superstrate's and substrate's conduction and convection properties, the cell temperature may run 15^{o} C to 40^{o} C (59^{o} F to 104^{o} F) above the ambient temperature. For example, a thick unfinned plastic substrate will cause the cell temperature to rise much higher than a thin aluminum finned substrate.

Because of the temperature rise, the module will suffer a voltage loss of approximately 2.2 mV/^{o}C (1.22 mV/^{o}F) rise in cell temperature. As an illustration, a hypothetical series-connected 30 cell 12 V, 0.5 A module rated at a cell temperature of 28^{o} C (82.4^{o} F) operates at a cell temperature 25^{o} C (77^{o} F) above the ambient temperature. Thus, for an ambient temperature of 28^{o} C (82.4^{o} F), the module will suffer a voltage loss of

$$30 \text{ cells} \times \frac{2.2 \text{ mV}}{{}^{o}\text{C cell}} \times 25^{o}\text{ C} = 1.7 \text{ V}$$

which is equivalent to a power loss of 0.85 W, or 14 percent. Conversely, if the ambient temperature falls below the rated temperature of the module, an increase in power will result. Although this is usually welcomed, it is occasionally detrimental. Certain inverters used to convert the dc photovoltaic output to ac require the output voltage to be no higher than a specified voltage. Therefore, choice of a particular module for a specific application should take into account the ambient temperature range at the site of application, the heat transfer characteristics of the module, and the requirements of the electrical system for the application.

The method of mechanically fastening the module in the array may either decrease or significantly increase the cell operating temperature. For a

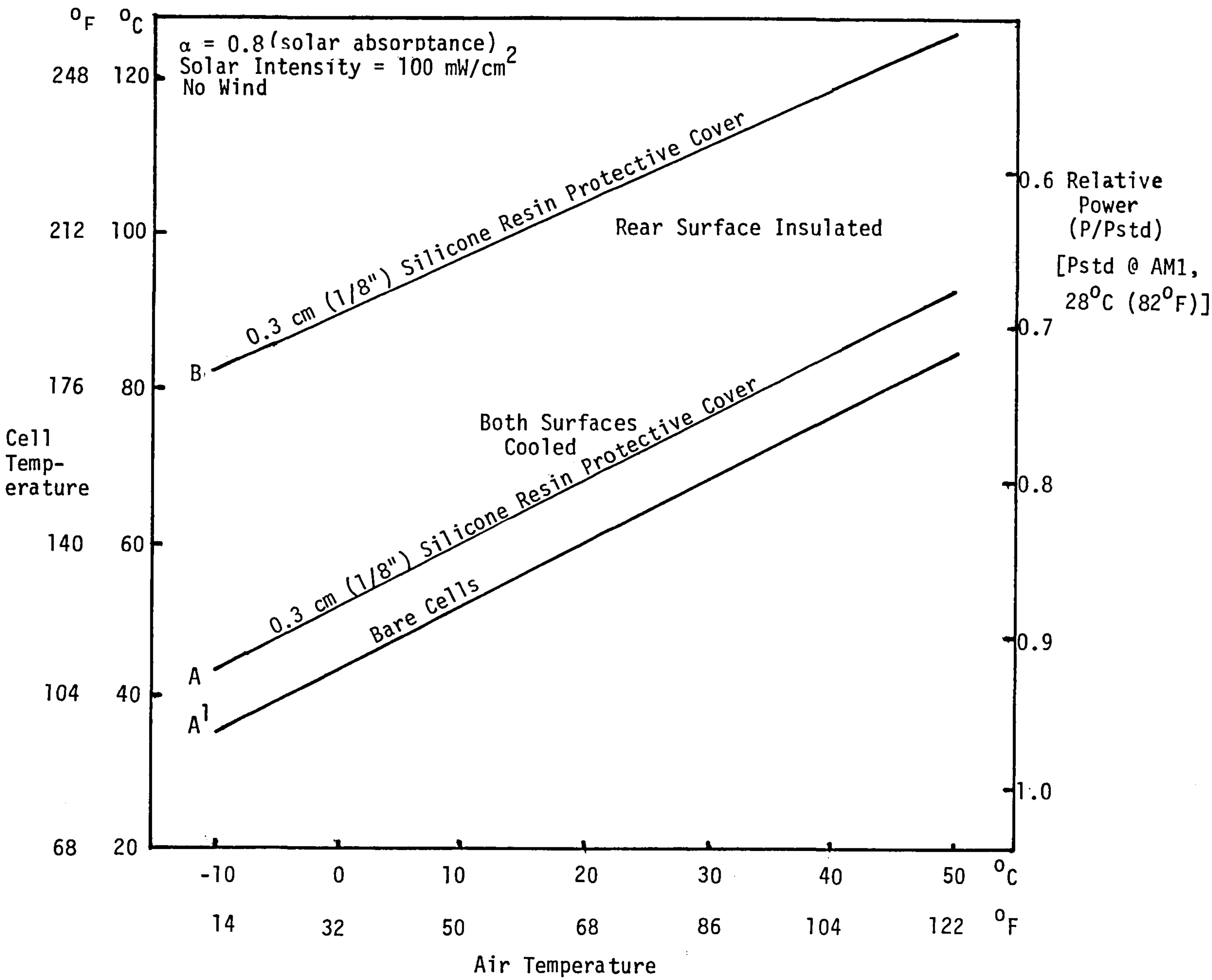


Figure 4. Module Thermal Characteristics

nonconcentrating photovoltaic array, it is uneconomical to have an active source of cooling such as forced air or water flow. The main source of cooling in most nonconcentrating photovoltaic panels is convection to the surrounding air. Therefore, any interference by the array support structure with the convection currents surrounding the module will affect the cell operating temperature. An example of a system where the support structure interferes with the module cooling is a structural system which uses a piece of wood or other material completely covering the back of the module to support the module. This could occur if the modules are placed directly on an already existing structure such as a slanted roof. The effect on cell temperature is shown in tests conducted by JPL and shown in Figure 4. The wind velocity is zero, but otherwise standard operating conditions are present. Line A represents the module with both sides open to the air, while Line B represents the module with only the front side open to the air. The module with the wood backing runs about 40° C (104° F) higher than the module with both sides unobstructed.

In the case of a module which has a transparent substrate and superstrate, a "greenhouse" effect may be created when the back is covered which will also significantly increase the cell temperature.

Therefore, the ideal array structure for minimizing the cell temperature will keep contact with the front and back surfaces of the module to a minimum. If contact must be made across the back surface, it should be made only across a small portion of the surface and with a highly conductive material, such as aluminum, which is itself open to the air on its own back surface.

3.2.2 Thermal Expansion

The design of the supporting structure of the array should take into account the dimensional change of the modules with temperature change. Material expands with change in temperature according to the equation:

$$\Delta = \alpha\ (T_1 - T_0)\ L$$

where

- Δ linear deformation (m) resulting from temperature change to T_1 (°C) from T_0 (°C)
- α coefficient of thermal expansion (°C^{-1})
- L original length of material (m)
- T_0 temperature (°C) at which material length is L

Figure 5 shows the range in values of α for materials used in module and array construction.

Generally, module thermal expansion has not been a problem with users of arrays of sizes up to 10 kWp. However, practically all the systems have been in use for only a few years. A potential problem exists with differential thermal expansion; the module may expand and contract at a different rate than the support structure. Depending on the strength of the module mounting structure and mechanical connector, the connector could break or the module mounting structure could crack, allowing the module to

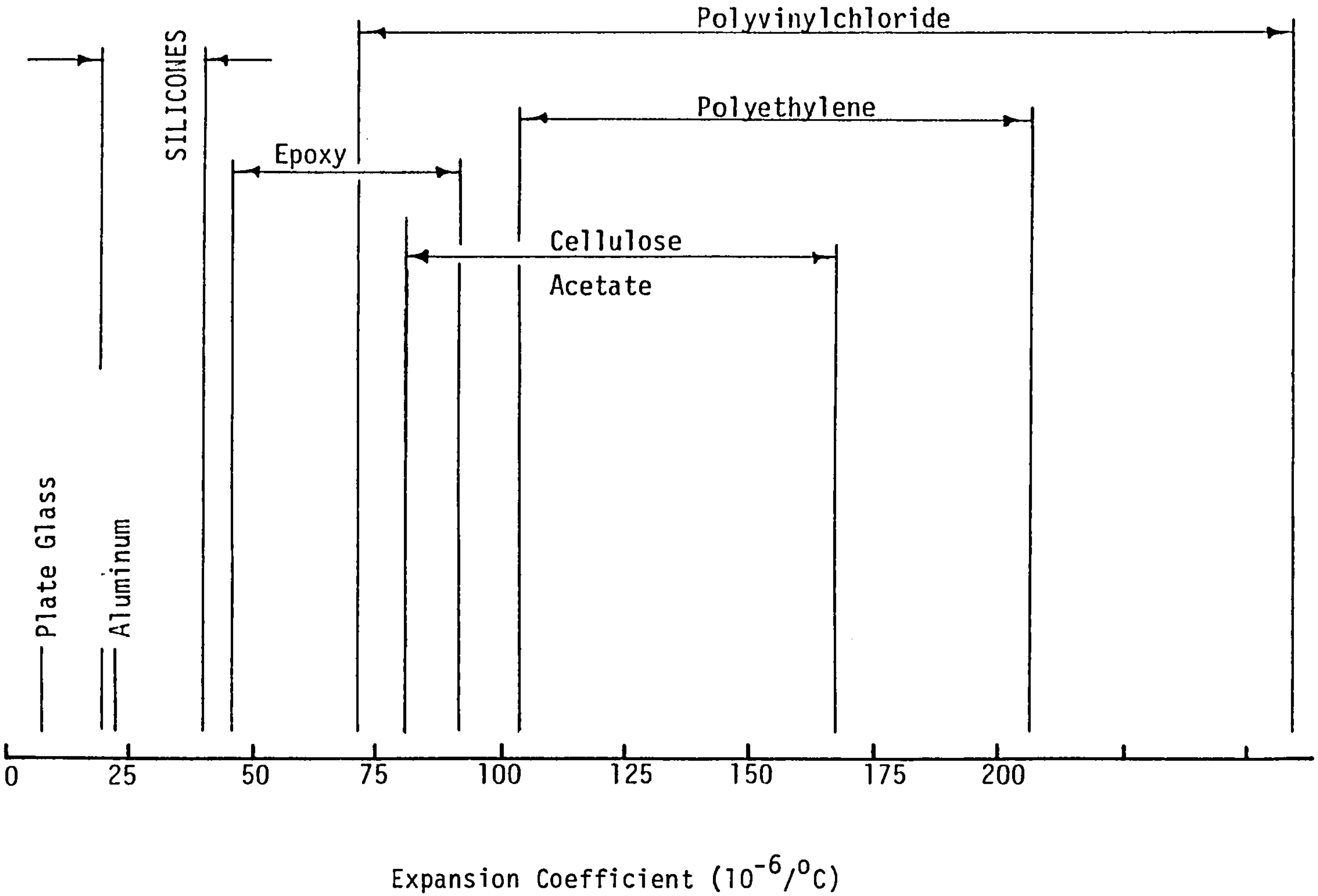


Figure 5. Module Differential Expansion

become disconnected from the support structure. In extreme environments or over a period of extended use, this differential expansion problem could become significant. Connection methods for taking this into account are given in Section 3.4.1.

Another potential problem exists with the module itself. Over a long period of time repeated temperature cycling may weaken the internal mechanism of the module causing delamination of the cells from the substrate. This may cause the cell operating temperature to increase to the point where the cell is damaged. In addition, cracking of the cells and/or fatigue of the cell-to-cell interconnects could result in open-circuiting of the module, particularly when there is a mismatching of thermal expansion characteristics of module materials. For these reasons, regular maintenance checks should be conducted on the modules.

3.3 Module/Array Mechanical Support Requirements

The two subjects covered in this section are module standards and module support requirements.

3.3.1 Module Standards

Private companies and governmental agencies have recommended that the uniform live load requirement for photovoltaic modules be a minimum of approximately 2400 N/m^2 (50 lb/ft^2) normal to the structure and that modules be capable of unimpaired operation under conditions of sustained twist of the mounting plane which causes deviation of approximately 2.1 cm/m (1/4 in./ft) from a true flat surface. The sales brochures of manufacturers of photovoltaic modules reveal that most of them are conforming to the recommended requirements. Fortunately this simplifies the design problems for the engineer concerned with designing the structure to support the modules. In reality the modules will rarely be exposed to pressures of 2400 N/m^2 (50 lb/ft^2). No United States building code would require the structure supporting the modules to sustain a 2400 N/m^2 (50 lb/ft^2) load except at extreme heights above the ground with an inclination greater than 30° from the horizontal. Nor would a well conceived structural support ever sustain distortions of 2.1 cm/m (1/4 in./ft). In most applications, a design load of approximately 960 to 1440 N/m^2 (20 to 30 lb/ft^2) normal to the surface would be adequate.

3.3.2 Basic Support Requirements

To understand the basic support requirements for photovoltaic modules, it is important to understand the function performed by each part of the array structure. Three basic definitions describe the functions covered in this handbook:

Beam: A structural member loaded along directions other than its axis.

Column: A structural member loaded along its axis.

Diaphragm: A planar structural element that ties together structural members joined at right angles or joined in parallel. The diaphragm prevents the structural members from rotating around the connection points.

The support requirements (illustrated in Figure 6) may be divided into module planar, module nonplanar, and foundation supports.

A. Module Planar Supports

First, the module is attached to structural members running along the top and the bottom edges of the module. These members are beams since they resist forces perpendicular to their axes at the connection points between the module and the members. A minimum of four connections of the modules to the beams are required; two on the upper beam and two on the lower beam. The module itself prevents the upper and lower beams from rotating around the connections, and is therefore a diaphragm. This increases the structural integrity of the array.

Secondly, additional structural elements are needed to span the distance between the beams. Since these new structural elements are subjected to axial loads at the connection points they are columns. For a two-tier structure, these new structural elements are subjected to both axial and perpendicular loading. They are beam-columns. These structural elements are designated as member AC' in Figure 6 for the two-tier structure, and as member AC in Figure 11 for the one-tier structure.

B. Module Non-Planar Supports

Structural elements are required to support the planar structure described in A above at a specified angle from the horizontal. These elements are columns and each element is designated as member BC in Figures 6 and 11. For the pole-mounted system, as shown by Figure 12, the functions of member BC are provided by the center beam and the angle adjustment member.

C. Foundation Supports

For nonportable systems, points A and B along the structure must be supported in the ground to prevent the structure from moving or overturning. Concrete foundations are used at these points.

For portable systems, points A and B along the structure may either be supported by heavy weights (sandbags, etc.), or the structure itself may be tied down securely to the ground.

The theoretical analysis of the basic elements described above is given in Section 4.5.2.

3.4 Module Installation and Removal

This section discusses the major mechanical and electrical considerations involved in module installation and removal from the array structure.

3.4.1 Mechanical Considerations

The main methods for attachment of the module to the array structure are welding, nailing, clipping, clamping, and bolting.

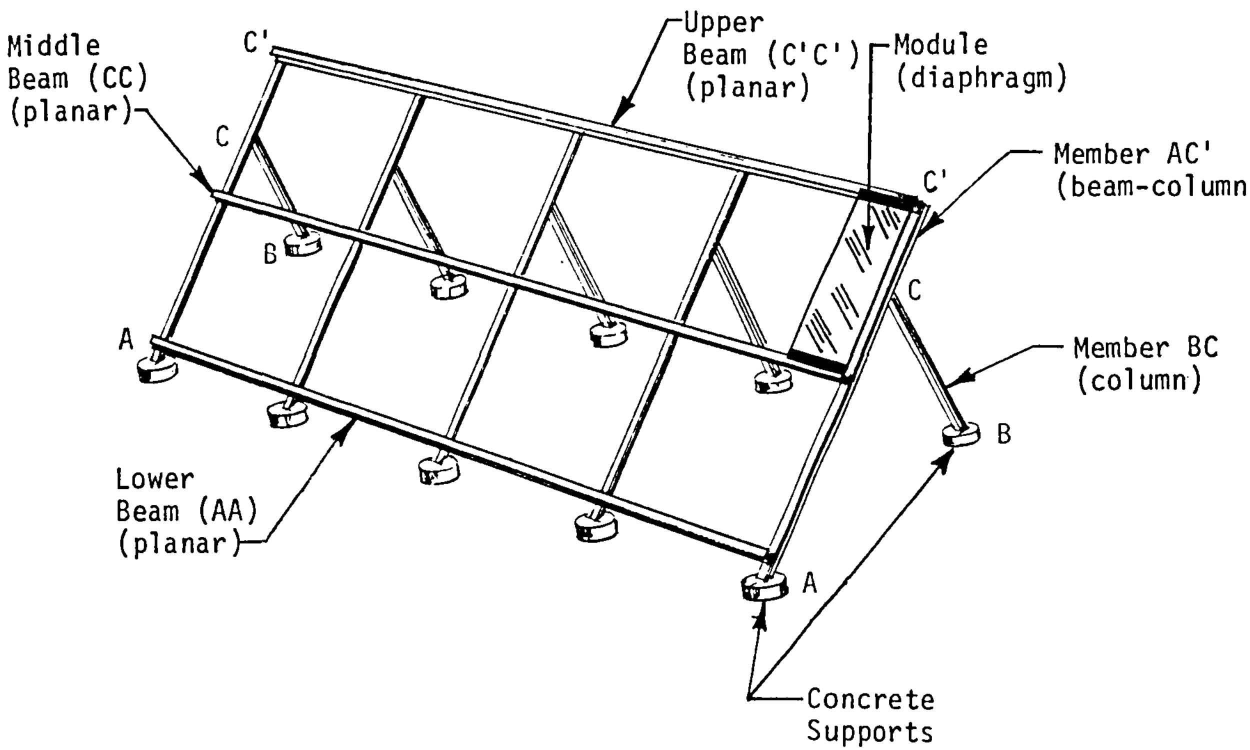


Figure 6. Module Support Requirements

Welding is not a recommended method for attachment to the array structure. There are two major problems. First, welding will produce a permanent connection. This will result in a major cutting operation each time a defective module must be removed from the structure. Second, the module frame and the array structure must be the same material. This restricts the potential materials to be used for the array structure, and so may increase the array cost. Also, it is very expensive and difficult to use the proper welding techniques in a field environment due to the potential heat damage.

Nailing is an optional attachment method. It is inexpensive, but there are two requirements for its use. First the module must have a suitable flange which will allow a nail to be driven through without cracking or other faults. Secondly, the structure must be wood. Damage to the delicate internal mechanism of the module may result from repeated blows on the module flange. From the removal point of view, there are two major problems. First, depending on the number of nails and their placement on the flange, removal of a module from the array may be a time-consuming task. Secondly, once the module is removed from the array structure, the existing holes in the structure may not be used because the module may work itself free under wind or snow loading. In addition several module installations and removals at the same location in the array may result in a permanently weakened structure.

Clipping and clamping are advantageous connection systems in terms of ease and expense of installing and removing modules. With special tools it is a simple process to clip or clamp the module to the array. The major problem with clipping and clamping systems is that they are the least reliable of the connection systems discussed in this section. Most clips and clamps provide a strong pressure attachment to the array parallel to the clip or clamp force. However, the only resistance to motion in the perpendicular direction is provided by friction. Because of this the structural system loses lateral support, and some structural pieces may have to be enlarged to account for possible load shift due to slip.

Bolting is the recommended method for mechanically attaching the module to the array structure. First, bolting provides a solid structural attachment in both the lateral and perpendicular directions. Second, bolts allow easy installation or removal of the modules. Third, they can be repaired or replaced easily, and are available essentially anywhere. Fourth, most module manufacturers provide bolt holes or slots in the flanges of their modules for attaching the module to the array structure. In addition to regular bolts, the designer may use the option of "torqueing" or "self-locking" bolts. Although more expensive than regular bolts they have an important advantage. They provide an interference fit to the structure without use of lockwashers or starwashers such that the bolts will not work themselves loose, yet permit snugging so as to allow for planar shift between the module and structure to accommodate thermal expansion mismatch due to different structural materials. The major disadvantage in using "torqueing" or "self-locking" bolts is that access must be available to both the top and bottom of the bolt for module installation and removal.

Several considerations are important when determining the mechanical connection type. The first is array structure penetration. It is necessary to provide penetrations into the array structure to allow the modules to be attached. Field drilling may be very expensive due to requirements for electrical power and labor costs. For these reasons, it is usually advisable to have the penetrations completed in a shop before being shipped to the site. However, tolerances must be closely controlled for incorrect placement of the penetrations may result in expensive field drillings to enlarge the holes. The second consideration is materials. If the array structure is steel, and the module frame is aluminum, the potential for an electrochemical reaction exists. In this case it is necessary to separate the two metals with a separator at all points of contact. The separating material may be a plastic, elastomer, or stainless steel. Because there is a potential for creeping and breakage in the plastic and elastomer separators, their use in certain environments may not be advisable.

3.4.2 Electrical Considerations

This section covers different generic considerations. Although it is beyond the scope of this handbook to describe detailed electrical schemes, a basic electrical connection system may be chosen for the array from this discussion.

Though photovoltaic systems are not explicitly mentioned in the National Electric Code (see Section 4.2) the various parts making up the photovoltaic system are covered. The following chapters of the code apply to a photovoltaic installation:

Chapter 1--General
Chapter 2--Wiring Design and Protection
Chapter 3--Wiring Methods and Materials
Chapter 4--Equipment for General Use
Chapter 6--Special Equipment
Chapter 7--Special Conditions
Chapter 9--Tables

In some installations used for research, development, or testing, the authority having jurisdiction for enforcement of the code may vary specific requirements in the code. Also, alternate methods may be permitted where it is assured that effective safety and maintenance procedures are maintained.

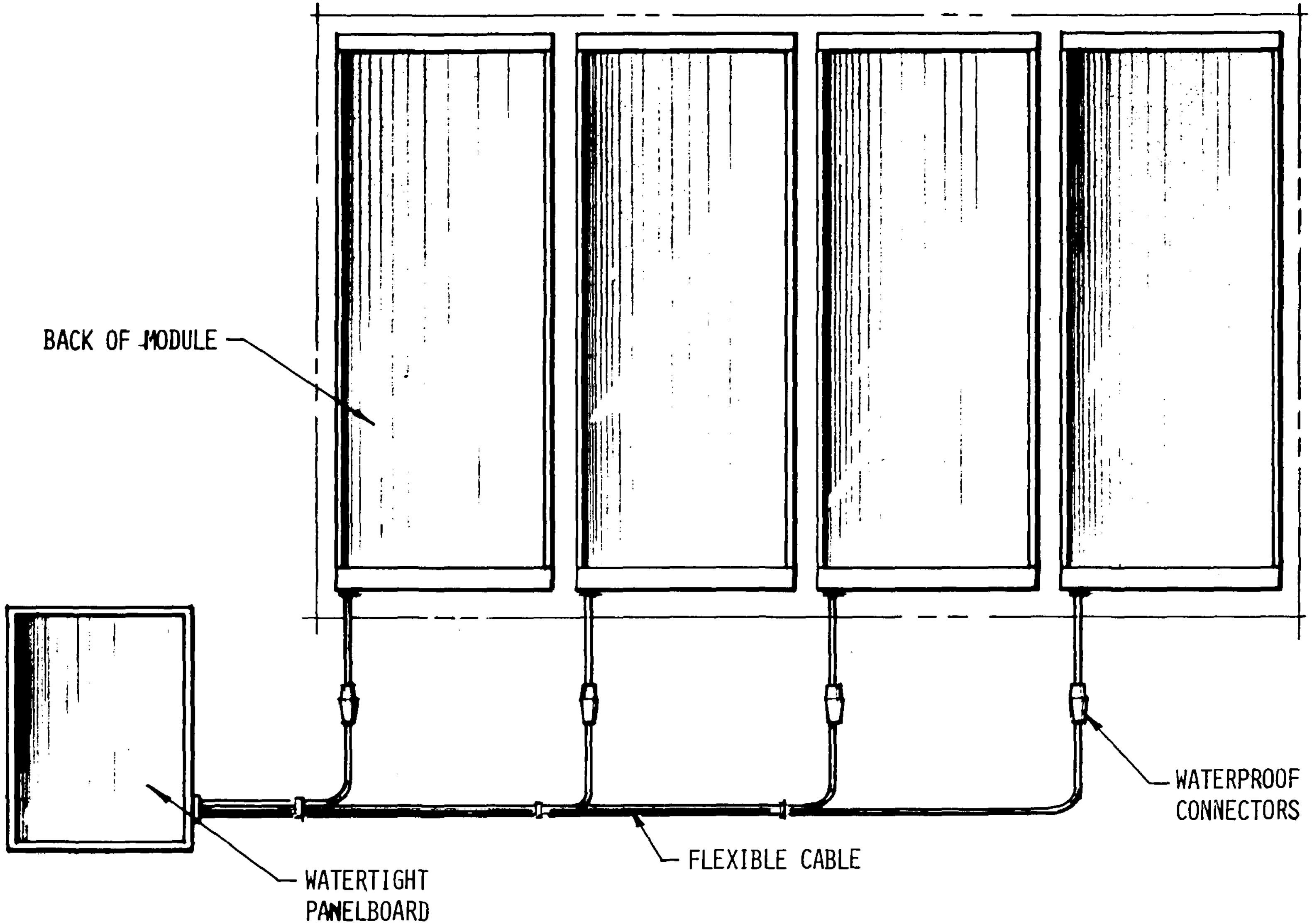
A. Electrical Connection

There are five major electrical connection designs covered:

Receptacle-plug (Figure 7A)
Junction box (Figure 7B)
Wireway-flexible conduit (Figure 7C)
Insulated cord-junction box-plug (Figure 7D)
Wireway-receptacle-plug (Figure 7E)

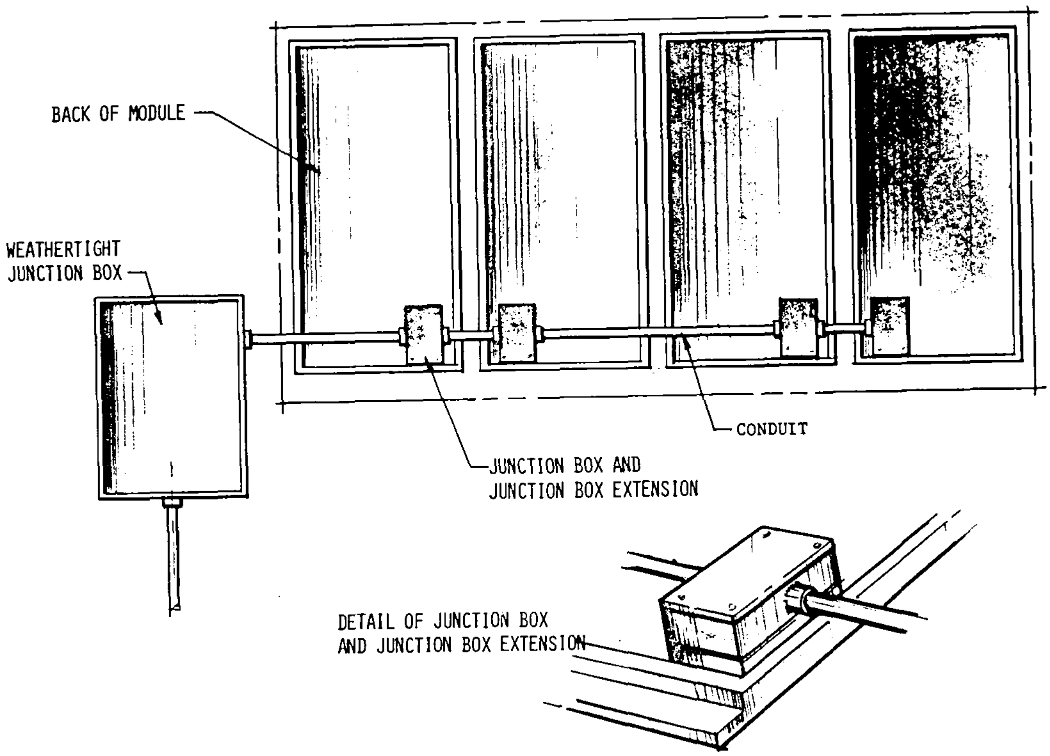
(a) Receptacle-Plug

The module is provided with a weatherproof plug and receptacle. Multi-conductor rubber cord shall run from a centrally located panelboard to each module and be connected with the plug and receptacle. The rubber cords, receptacles and plugs shall be designed for outdoor use. The rubber

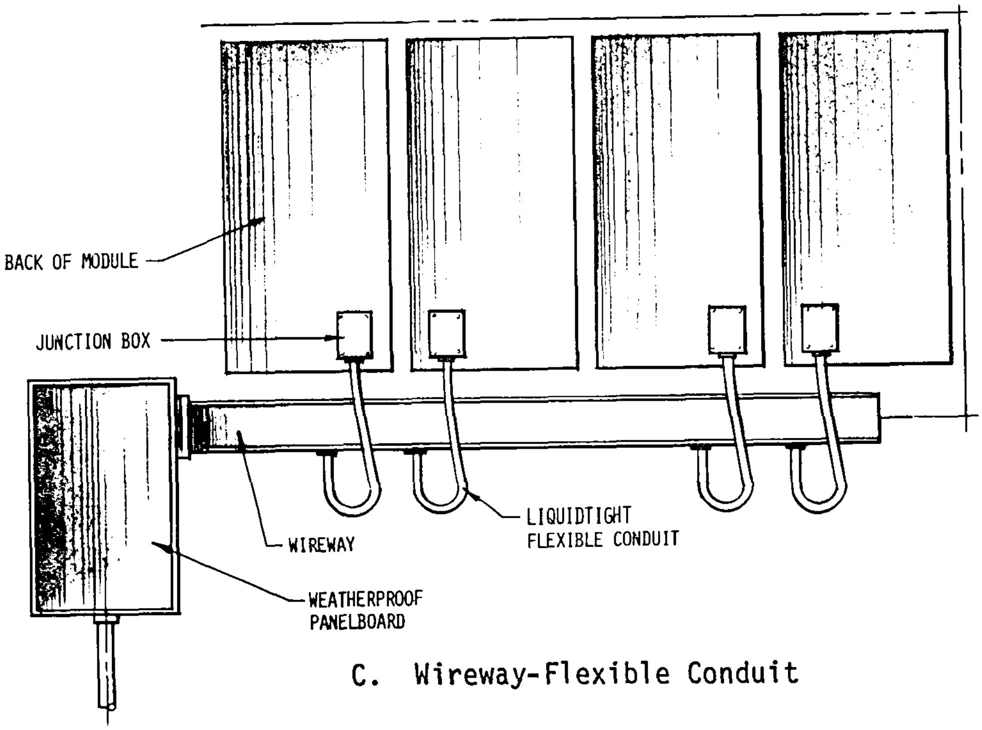


A. Receptacle-Plug

Figure 7. Electrical Systems

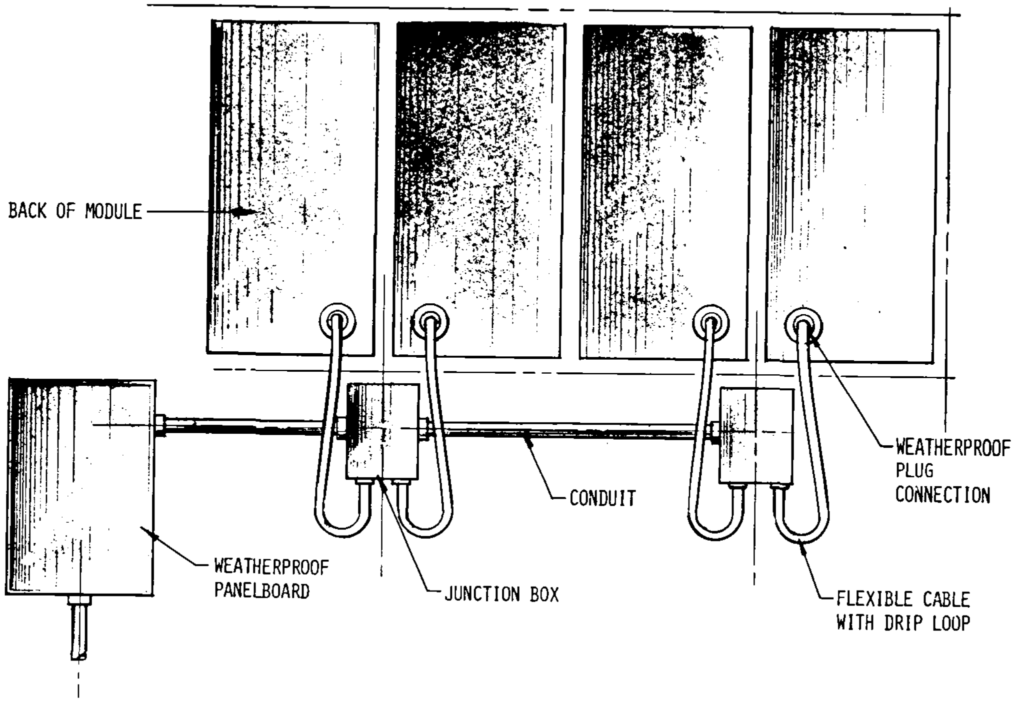


B. Junction Box

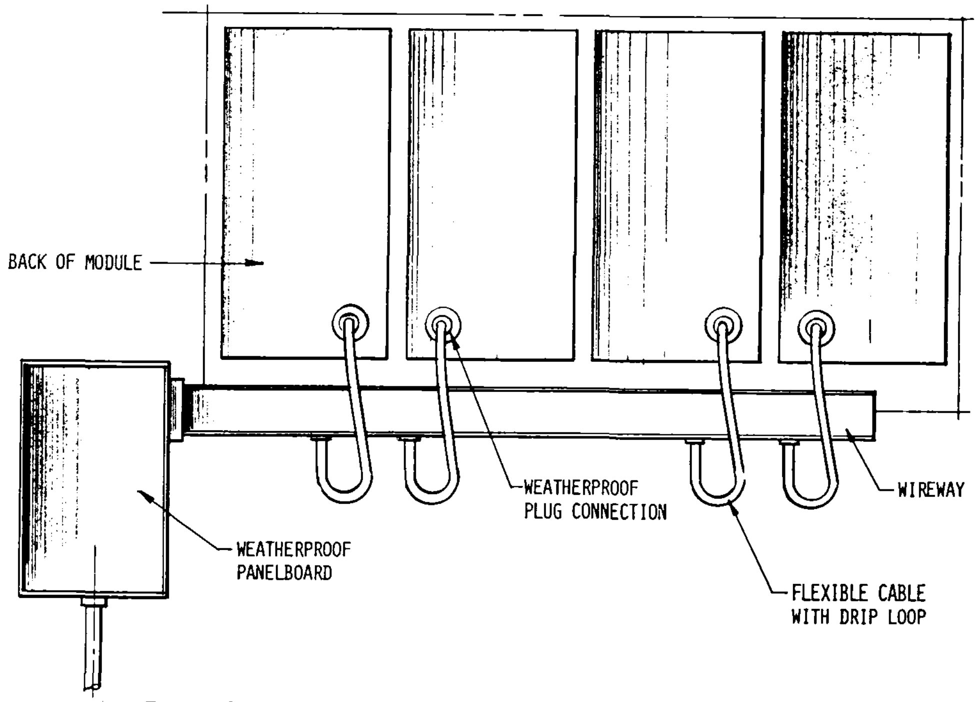


C. Wireway-Flexible Conduit

Figure 7. Continued

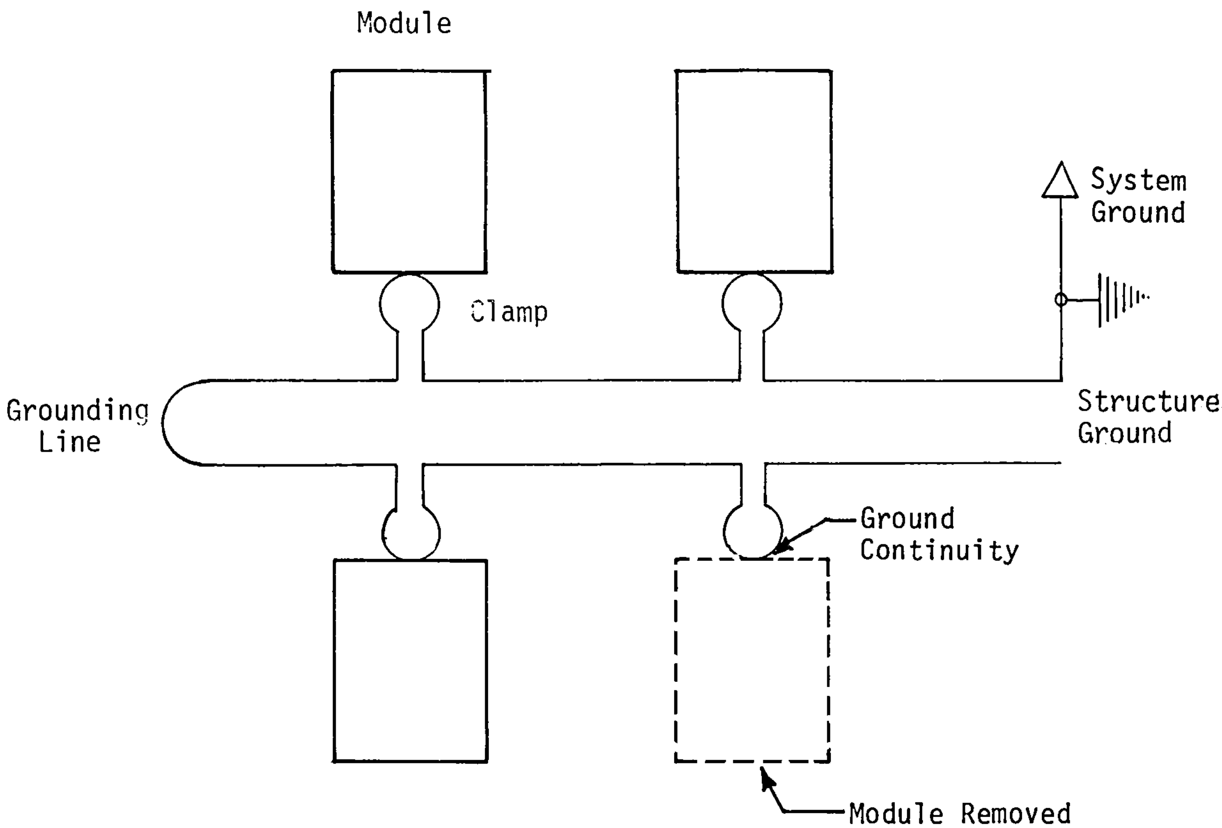


D. Insulated Cord-Junction Box-Plug

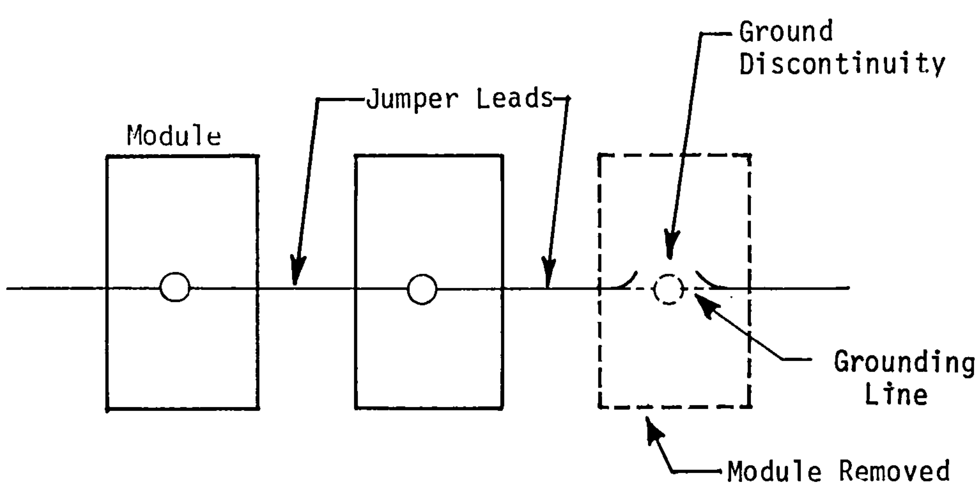


E. Wireway-Receptacle-Plug

Figure 7. Continued



F. Electrical Grounding System #1



G. Electrical Grounding System #2 (not recommended)

Figure 7. Concluded

cord shall be strapped, supported and arranged to prevent sagging. The rubber cord shall be installed and attached to the array structure to prevent damage to the wire. A separate ground wire, connected to each module frame, junction boxes, panelboards and the array structure, shall be installed to insure that ground continuity is maintained.

Conduit raceways shall be used for panelboard feeder circuits and any underground wiring. Junction boxes shall be installed where necessary.

(b) Junction Box

The module or panel are delivered to the site with factory installed junction boxes. After the modules or panels are attached to the supporting structure, the junction box coverplate is removed and a junction box extension is attached to the module junction box. Conduits run between and are attached to the junction box extension, all along the back of the array. Modules can be removed from the front of the array by disconnecting the splice in the junction box, and removing the four screw connections between the junction box and the junction box extension without disturbing the conduits.

Splices between branch circuit wiring and module or panel are made inside the junction box and box extension with approved type electrical wire nuts.

The raceways shall be rigid galvanized or flexible conduit or electrical metallic tubing. Junction box extensions, in order that this system not be overly expensive, should be plastic or some other inexpensive nonmetallic material. All conduits are strapped to the array support structure.

(c) Wireway-Flexible Conduit

The modules are delivered to the site with factory installed junction boxes. Liquid tight flexible metal conduit shall connect to the junction boxes with weatherproof connectors on the module end and connect to a 10 cm by 10 cm (4 in. by 4 in.) weatherproof wireway on the other end.

(d) Insulated Cord-Junction Box-Plug

The modules or panels are delivered to the site with factory installed weatherproof receptacles and plugs. For safety, the receptacles and plugs shall have no exposed contacts when separated. The rubber cord shall be as short as possible, but with a drip loop and shall be connected to the bottom of a junction box with a weatherproof connector.

The raceways shall be rigid galvanized or flexible conduit or electrical metallic tubing and shall be run along the back of the array and be fastened to the supporting structure. Each raceway shall terminate in a weatherproof junction box with a gasketed cover. Removal of a single module or panel from the front of the array shall not disturb the conduits and junction boxes which are located on the back of the array structure. All raceways and junction boxes shall be installed in the field.

(e) Wireway-Receptacle-Plug

The modules are delivered to the site with a weatherproof receptacle and plug. A grounded rubber cord and plug shall connect to each module receptacle and shall connect to the bottom of a wireway with a weatherproof connector.

The raceway shall be a 10 cm by 10 cm (4 in. by 4 in.) weatherproof wireway with a screw cover. The wireway shall run from the centrally located panelboards, along the back of the array and shall be attached to the array structure. Splices shall be made in the wireway. Multi-conductor rubber cord with a weatherproof plug shall connect to the module.

B. Safety Considerations

Recommendations by several organizations including NASA and the National Fire Protection Association, regarding general safety practices include the following three factors:

(a) Voltage Maximum

It is recommended that the voltage of any integral group which may have electrical maintenance work performed on it be no greater than 50 V.

(b) Grounding

It is recommended that all modules having metallic substrates, supports, or frames be provided with an external grounding termination meeting the following requirements: (1) Solder attachment shall not be allowed; (2) The ground circuit shall not be made through a connector interface; (3) A terminal shall be provided which accepts a ground wire equal or larger in gage than accepted by the module output terminals; (4) Grounding circuit shall not be provided by a Vendor-installed pigtail; (5) If output terminations are provided in a J-Box, the ground termination should also be located internal to the J-Box with suitable gland or compression type penetrations provided; (6) The ground terminal shall not be used for any other purpose (i.e., module assembly or installation). All exposed metallic or conducting surfaces shall demonstrate electrical continuity to the external grounding connection; and (7) the array structure itself should not be used for grounding.

The suggested method of grounding the array is shown in Figure 7F. Here, each module is attached by a clamp or similar device to a small loop coming from the main ground line. The advantage of this system is that when a module is removed, only the removed module ground is defeated, not the entire system ground. This problem is illustrated in the grounding system of Figure 7G, where removal of one module will result in the system ground being defeated. Such a system is not recommended. See also Article 250 of the NEC for more details on grounding.

(c) Lightning Protection

A lightning rod(s) should be erected in accordance with the specifications given in Article 280 of the NEC.

3.5 Array Layout

3.5.1 Factors Affecting Packing Efficiency

Module packing efficiency, the efficiency with which the area available for the photovoltaic array contains photovoltaic modules, must be considered during the design phase of the photovoltaic system. Shading of one row (planar element) of modules by another row, and space considerations are the major considerations when arranging the modules. Other limitations are transportation capability (limited by truck size), maintenance accessibility, and accessibility to sunlight.

Shading of the cells within the array will cause power loss for the following four reasons:

- The amount of radiation each cell receives is reduced so that the cell output is correspondingly reduced.
- In a series string, the shaded cells may block the current flow of the illuminated cells.
- In a parallel string, the shaded cell may shunt part of the generated current of the illuminated cells.
- Shaded cells may become reverse-biased by the voltage developed across them.

For these reasons, shading of the array should be kept to a minimum. One method of doing this is to space the rows in the North-South direction so that the modules will either never be shaded, or shaded for only short periods in the early morning and late afternoon. Although there is no established rule-of-thumb in this area, one approach is to choose the spacing between rows such that there will be no shading of the rows after 0900 hours on December 22 for sites located in the Northern Hemisphere (after 0900 hours on June 22 for sites located in the Southern Hemisphere). This approach will result in negligible shading of the array through the year. If site space limitations are tight, and the user is willing to accept greater array losses due to shading longer in the morning and earlier in the afternoon the spacing could be determined for a later time, or for a different time of the year. In addition, the array should be oriented so that it faces due south for Northern Hemisphere sites and due north for Southern Hemisphere sites.

Basic parameters needed to determine row spacing are the sun's altitude (SALT) and the sun's azimuth (SAZM) angles for the day of the year and time of day that the designer has established as the criterion for no shading. These parameters, which describe the position of the sun in the sky with respect to the site location are illustrated in Figure 8A.

SALT and SAZM may be determined for any instant in time when the latitude (l) and hour angle (h) for the site, and the declination angle (d) of the sun are known. The latitude is the location of the site north or south of the equator and is expressed in degrees North Latitude (positive) or degrees

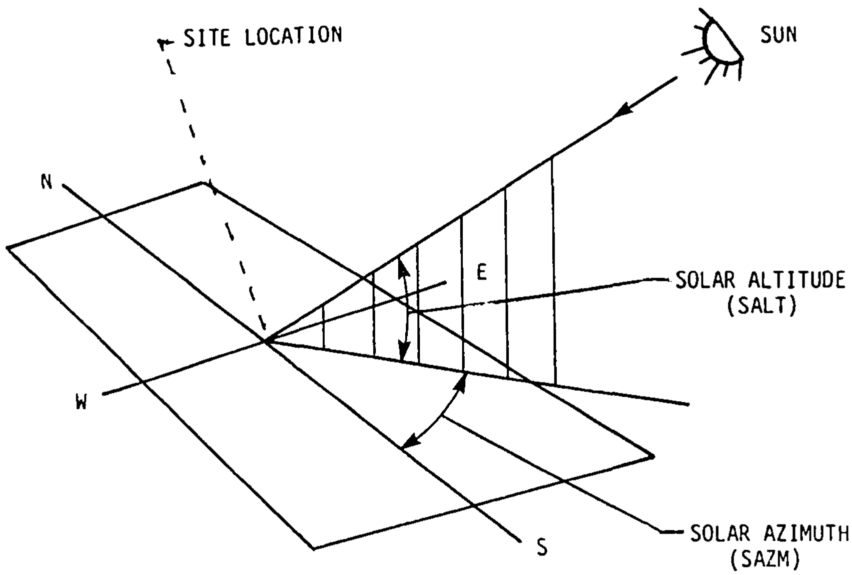
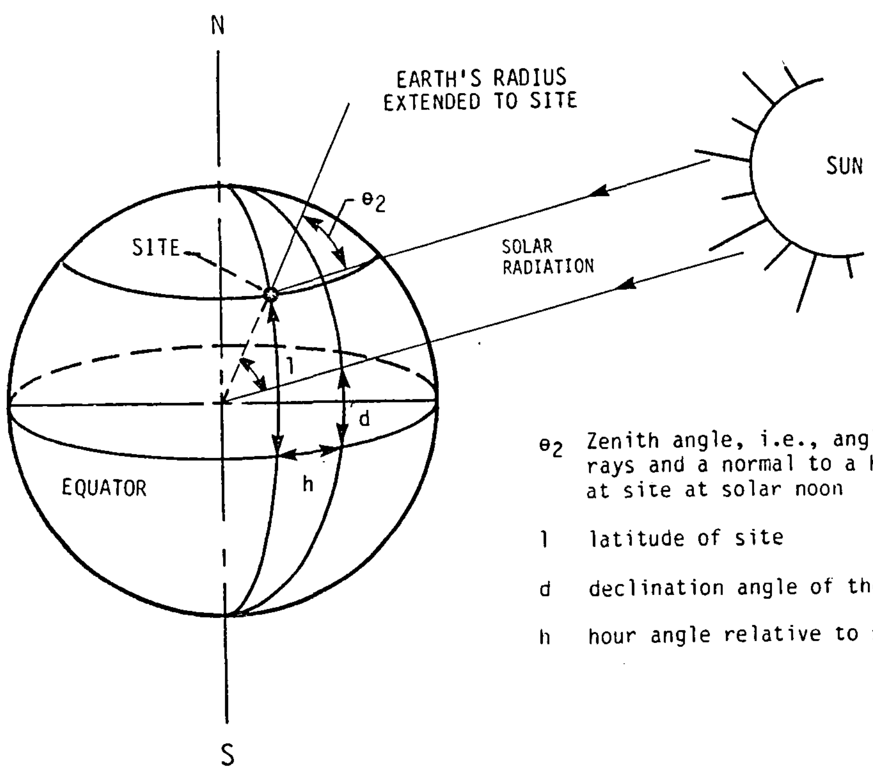


Figure 8A. Site Location Relative to Solar Altitude and Solar Azimuth



θ_2 Zenith angle, i.e., angle between sun's rays and a normal to a horizontal surface at site at solar noon

l latitude of site

d declination angle of the sun

h hour angle relative to site

Figure 8B. Sun's Position in Sky Relative to Earth Locations

South Latitude (negative). The hour angle expresses the time of day at the site in solar time. The hour angle is zero at solar noon, i.e., when the sun is at its zenith, and each hour of departure from solar noon is $360^{o}/24$ or 15^{o}. Hour angles from 0^{o} to 180^{o} before solar noon are negative, while hour angles from 0^{o} to 180^{o} after solar noon are positive. The declination angle is the angular position of the sun north (positive) or south (negative) of the equator. These parameters are shown in Figure 8B.

The declination angle varies with time of year and slightly from year to year for the same calendar day. For purposes of this handbook, however, the declination angle for any given day is considered constant and the value from one year to the next is considered the same for the same calendar day.

When the latitude and longitude for the site are known, use the equations and Tables IIA and IIB below to calculate the following:

Hour Angle for the Site

$$h = 15\ (t - 12 + TZN + ET) - Long \qquad (1)$$

where

h	hour angle of the site relative to solar noon
t	local clock time (hours after midnight) chosen to avoid array shading in the morning
TZN	time zone number for the site (see Table IIB)
ET	equation of time (see Table IIA)
Long	longitude of site (West is positive; East is negative)

$$h' = \cos^{-1}\ (-\tan Lat)(\tan d) \qquad (2)$$

where

h'	hour angle of the site relative to solar noon at sunrise
Lat	latitude of the site (North is positive; South is negative)
d	declination angle of the sun (see Table IIA)

If $|h| > |h'|$, then the time chosen to avoid shading occurs before sunrise at the site.

Sunrise Time at the Site

$$SRT = 12 - \frac{h'}{15} - ET - TZN + \frac{Long}{15} \qquad (3)$$

where

TABLE IIA. THE SUN'S DECLINATION AND EQUATION OF TIME *
[d is the declination angle of the Sun; ET is the Equation of Time.]

Day	1		8		15		22	
Month	d, deg	ET, hr	d, deg	ET, hr	d, deg	ET, hr	d, deg	ET, hr
January	-22.99	-0.061	-22.21	-0.113	-21.08	-0.158	-19.63	-0.194
February	-17.04	-0.189	-14.92	-0.237	-12.60	-0.236	-10.12	-0.226
March	- 7.51	-0.206	- 4.81	-0.181	- 2.06	-0.149	- 0.71	-0.115
April	4.62	-0.065	7.28	-0.031	9.83	-0.001	12.26	0.025
May	15.13	0.049	17.14	0.059	18.91	0.062	20.42	0.057
June	22.08	0.038	22.86	0.018	23.32	-0.006	23.44	-0.031
July	23.10	-0.063	22.45	-0.083	21.50	-0.098	20.24	-0.106
August	17.97	-0.104	16.09	-0.093	13.99	-0.074	11.72	-0.047
September	8.22	0.001	5.62	0.039	2.96	0.080	0.25	0.122
October	-3.26	0.172	-5.95	0.207	-8.58	0.237	-11.11	0.259
November	-14.50	0.273	-16.62	0.270	-18.53	0.256	-20.17	0.231
December	-21.83	0.182	-22.74	0.134	-23.28	0.081	-23.44	0.023

* Taken from The Ephemeris 1981, U.S. Navy.

Table IIB. TIME ZONE NUMBERS FOR THE WORLD FOR STANDARD TIME

TIME ZONE NUMBER	STANDARD MERIDIAN
-12	180° E.
-11	165° E.
-10	150° E.
-9	135° E.
-8	120° E.
-7	105° E.
-6	90° E.
-5	75° E.
-4	60° E.
-3	45° E.
-2	30° E.
-1	15° E.
0 (Greenwich)	0°
+1	15° W.
+2	30° W.
+3	45° W.
+4 (Atlantic)	60° W.
+5 (Eastern)	75° W.
+6 (Central)	90° W.
+7 (Mountain)	105° W.
+8 (Pacific)	120° W.
+9 (Yukon)	135° W.
+10 (Alaska-Hawaii)	150° W.
+11 (Bering)	165° W.
+12	180° W.

NOTE: 1. Negative (-) TZN is the number of hours later than Greenwich Standard Time; Positive (+) TZN is the number of hours earlier than Greenwich Standard Time.

2. The Standard Meridians given are the approximate centers of the time zones.

3. ( ) Indicate names given to some of the Standard Time Zones.

SRT sunrise time at the site (clock time)*

Altitude Angle of the Sun

$$SALT = \sin^{-1} (\cos Lat \cos h \cos d + \sin Lat \sin d) \quad (4)$$

where

SALT altitude angle of the sun with respect to a horizontal plane which passes through the site location

Azimuth Angle of the Sun

$$SAZM = \sin^{-1} (\cos d \sin h/\cos SALT) \quad (5)$$

where

SAZM azimuth angle of the sun with respect to solar South (west of South positive; east of South negative)

The following equations and Figure 8C may be used to determine the spacing between the rows:**

$$SPACE_1 = \frac{Y \times \sin(\delta + \tau)}{\sin(\delta)} \quad (6)$$

$$SPACE_2 = \frac{X \times \sin(\delta + \tau)}{\sin(\delta)} \times \frac{1}{\sin(\tau) \times \cot(SALT) \times \sin(a)} \quad (7)$$

$$a = |SAZM + CAZM|$$

$$\delta = \arctan\left[\frac{\tan(SALT)}{\cos(a)}\right]$$

where

$SPACE_1$ distance required between the front supports of a row and the row directly behind it in order to ensure that the shadow will be below the array row

*In many localities, clocks are advanced one hour beyond Standard Time in summer. In the U.S., such time is referred to as "Daylight Savings Time". In such cases, add 1 hour to SRT.

**From "Optimized Spacing between Rows of Solar Collectors," p. 3-15, in the Proceedings of the 1977 Annual Meeting of the American Section of ISES, Orlando, Florida, 1977.

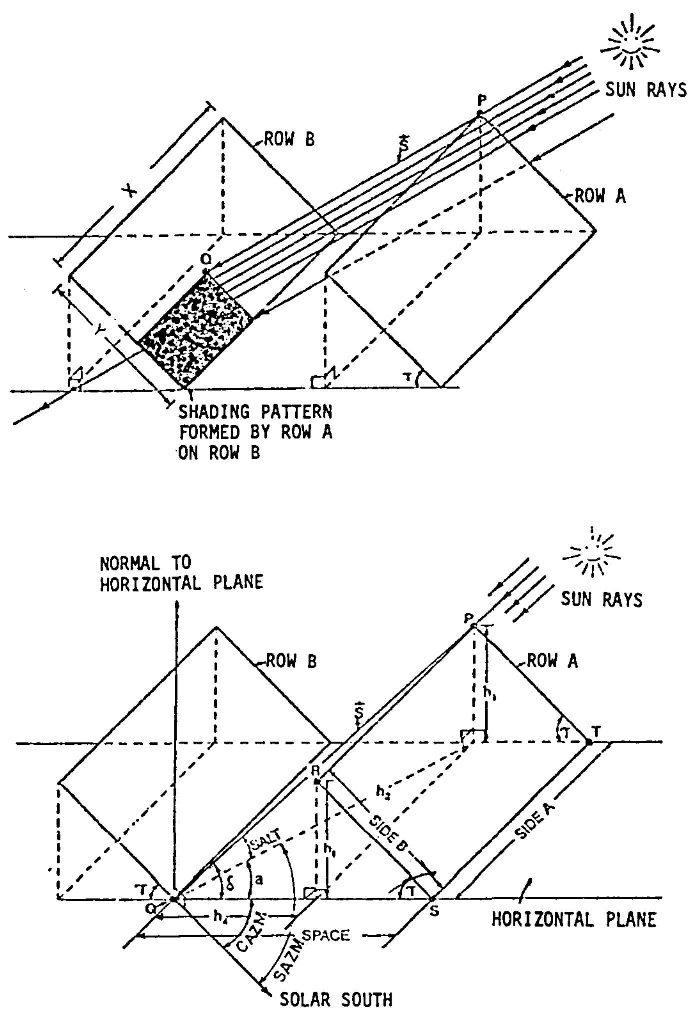


Figure 8C. Array Layout to Avoid Shading

$SPACE_2$	distance between the front supports of a row and the row directly behind it in order to ensure that the shadow will be to the left (morning) or right (afternoon) of the row
δ	projected altitude angle of the sun on a plane perpendicular to the rows
α	angle between the sun rays projected on the horizontal plane and a plane perpendicular to the rows
τ	tilt angle of the array (known as θ in Figure 13)
SALT	solar altitude at the site for the date and time chosen (calculate using Equation (4) above, or obtain data from NOAA or other sources)
SAZM	solar azimuth at the site for the date and time chosen (calculate using Equation (5) above, or obtain data from NOAA or other sources)
CAZM	collector azimuth (positive when measured west of solar south for both North and South Latitudes)
X	width of row between left-most and right-most support points
Y	height of row (measured along the plane of the row)

To determine the spacing, use the following steps:

Step 1. Solve for $SPACE_1$ in Equation (6)
Step 2. Solve for $SPACE_2$ in Equation (7)
Step 3. Since either $SPACE_1$ or $SPACE_2$ guarantees no shading for the date and hour selected, the smaller of these values is the recommended spacing between the rows.

Another method that may be utilized to reduce shading (reduce row spacing) is to step the rows, i.e., step the array rows by a fixed delta increment in height. For Northern Hemisphere sites, increase row heights in the south to north direction and for Southern Hemisphere sites, increase row heights in the north to south direction.

3.5.2 Structural Arrangements

The following principles should be used in layout of the array structure:

A. As far as practical, modules should be supported in one plane (row) to minimize the structure and foundation costs.

B. Structural column members should be, as far as practical, maximized in number in the design in comparison to beams. This is because members are more efficiently used when loaded axially than when loaded nonaxially.

C. Where practical, use long supporting beams supported at four or more locations (three span minimum). This creates a statically indeterminate beam which will result in smaller moments than a comparable simple span or two-span beam.

D. Field connections, such as beam splicing, back-to-back connection of channels to form beams, welding of clips, plates or fittings to beams, etc., should be avoided whenever possible, i.e, do as much as possible in the shop.

E. To ensure simple maintenance procedures, the design should stress simplicity in structural arrangement.

4.0 ARRAY STRUCTURE DESIGN

Normally, array structure design is performed by a professional structural engineer. This handbook, and in particular, this section has been written to limit the need for a structural engineer in designing array structures for a selected range of array sizes in either of two selected structural systems. It is however necessary, for the user of this handbook to have a rudimentary knowledge of statics and material strengths so that the intent and limitations of the handbook are understood.

4.1 Philosophy of Structural Design

Structural design can be reduced to a two-part process, the first being analysis and the other being design. Analysis is the process of applying the laws of physics and the resolutions of environmental loads to a conceptual model of the structural system being considered. Design is the process of using the laws of engineering mechanics of deformable bodies in order to determine the size, shape, and material of the members that will compose the finished product. The end product will be a structure that should satisfactorily serve its purpose over its expected life.

Fundamental to the analysis portion of structural design is the determination of all significant loads (forces) applied to the structure. Loads generally are separated into two broad categories: dead loads and live loads. Dead loads are the weights of the structural elements plus all other materials permanently supported by the structure. For the purposes of this handbook these materials consist of the photovoltaic modules, electrical and mechanical hardware, conduit, and all other permanently installed equipment that may be required for the particular array. Live loads generally constitute all other applied loads which include service and maintenance loads, and temporary wind, seismic (ground motion due to earthquakes or volcanic action), snow and ice loads. However, the primary structural manuals define the live load as not including the earthquake or wind loads, and this more restrictive definition will be used in this handbook.

Building codes generally dictate what are and are not to be considered live loads. They will also specify how all loads are to be applied to the structure. These loads as addressed by the various United States' building codes do not directly apply to photovoltaic array structures. Therefore, the design engineer must exercise judgment in interpreting the codes and what loads and their magnitude to apply to the structure.

The wind load is the major design load factor in the design of a photovoltaic structure. United States' building codes are designed to apply to building structures which have a life expectancy in the range of between 30 and 75 years. Wind loads as found in building codes are based on a 50-year mean recurrence interval for extreme wind velocity including gusting. Statistically this means that there is a 2 percent probability of the design wind velocity (hence, the design wind load) being exceeded in any one year. The design engineer should determine the anticipated useful life expectancy of the structure along with its acceptable risk factor (which may differ from the risk factor of the photovoltaic module) before determining the wind load. Determination of acceptable risk should include consideration of both economic loss and human life loss if failure of the structure occurs during its life. In lieu of a rigorous risk analysis, it is recommended that a 50-year mean recurrence interval be used.

Other environmental loads consist of seismic loads and snow and ice loads. Seismic loads are generally not significant and can usually be neglected when designing array structures. Snow and ice loads on the other hand may be significant, depending on the site location.

After determination of the environmental loads, the type of generic structural system would normally be developed. Primary alternatives include a single-pole supported system, a beam-and-post type system with either a rigid or braced frame, or a triangular frame. This handbook limits its scope to triangular frames and single-pole supported systems (see Section 4.5). Selection of a generic structural system contained within this handbook can be made, due to the limitations, before the loads are determined.

The triangular frame has special advantages over most other generic framing systems. Its members are slanted and may be positioned at the desired angle of inclination for optimum exposure of the modules to the sun. In addition, it is a highly efficient structural system, and it is readily adaptable to a variety of sites. The disadvantage of the triangular frame is that it may consume more site area than is desirable due to the distance between its two support points. The pole-supported system lacks the efficiency and flexibility of the triangular frame but may be more cost-effective and require less room, especially for smaller arrays.

Considerations should not be limited solely to structural efficiency when selecting a generic structural system. Other considerations, some of which are in the realm of the architect, but which should be considered in the absence of an architect by the engineer are:

A. <u>Aesthetics</u>

The structural and array should conform with the motif of the surrounding natural environment or architectural motif of the urban area and property onto which the array is to be placed.

B. <u>Maintenance</u>

Consideration should be given to the following factors:

- Will maintenance loads be placed on the structure?
- What type of upkeep and inspection will be needed during the life of the structure?
- Is the structure designed for easy maintenance and service?

C. <u>Material Availability</u>

In some parts of the world, wood is non-existent and aluminum and concrete have both been in short supply in the United States at various times. Therefore, the engineer should inquire about the availability of material and structural shapes before finalizing the design. It is not uncommon to discover after designing a structural system that certain materials or structural shapes are not available locally or, if available, that there is an excessive lead time required for delivery to the construction site.

D. Costs

The engineer invariably faces the question of the cost tradeoff between material cost and labor cost. As a general rule in the United States, labor is the larger contributor to cost. Therefore, every attempt should be made to limit the labor requirements at the construction site, i.e., have as much as possible of the fabrication and construction done in the shop and not at the site. In other areas of the world, particularly the Third World countries where their economies are labor intensive, attempts should be made to economize on materials in order to provide more on-site labor. The engineer must also keep in mind the level of skill of local labor.

There are basically two types of design approaches in terms of the safety factor used in the United States; the working stress method and the ultimate strength method. The working stress method is used for all the materials in this handbook except concrete which uses the ultimate strength method. The working stress method applies a safety factor to the allowable stress that the material may be subjected to while applying actual loads to the material. The ultimate strength method applies a safety factor to the loads (by increasing the design load) while designing the material to its ultimate stress capacity.

Finally, as discussed in Section 3.5, consideration must be given to the array layout and structural arrangement so as to minimize land usage, yet avoid shading of the array.

4.2 Codes

Throughout the world there are local zoning and building codes to which the photovoltaic array designer must conform. Armed with these codes, localities exercise police powers over owners, consultants and contractors who design and construct buildings and other structures. This control extends over all phases of design and construction and include land use, design specifications, and construction methods. To ensure compliance, field inspections are made by local officials.

As the first design step, the designer of a photovoltaic array should consult the local zoning and building codes.

The zoning code will establish whether the anticipated land use is permissible. Provisions of the local zoning codes, though not explicitly addressed to photovoltaic array structures, are applicable in their design. For example, setback requirements from property lines will affect the array site layout. Also, many communities have planning commissions which recommend whether or not building permits will be issued. Finally, an environmental impact statement may have to be filed for the array.

The building codes establish the design criteria for the structure to insure that the structure will be safe and will be adequate to sustain the expected loads over the expected life of the structure.

The local building code will often be adapted, in part or whole, from one of the nationally recognized codes in the U.S. Among the most often used nationally recognized codes are the following:

A. Uniform Building Code (UBC): International Conference of Building Officials (1976 Edition). (See Section 7.6.)

B. National Building Code (NBC): American Insurance Association (1976 Edition).

C. Southern Standard Building Code (SSBC): Southern Building Code Congress (1977 Edition).

D. Basic Building Code (BBC, also known as the BOCA Code): Building Officils and Code Administrations International, Inc. (1978 Edition).

E. National Electrical Code (NEC): National Fire Protection Association. (Also of interest are the National Electrical Code® Handbook: National Fire Protection Association and National Electrical Code® Reference Book: Prentice Hall) (1978 Edition).

The UBC is the primary structural code for this handbook. Other structural codes are secondary references. The NEC is the primary electrical code.

4.3 Load Factors

The design loads for the array structure may be divided into the following groups: dead loads, live loads, wind loads, seismic loads, snow loads, thermal loads and miscellaneous loads. Where there are no applicable local codes, the following minimum requirements shall be met.

4.3.1 Dead Loads

Dead loads are those loads permanently attached to the array structure. Examples are the weights of the modules, electrical and mechanical connections, conduits and other hardware, extraneous hardware and the structure itself.

Usually the manufacturer supplies the weights of the module and electrical and mechanical subsystems. Structural material weights for various sections are generally available as part of their designation system. In lieu of such explicit designations, the following material densities may be used to compute the weight of the materials:

- Steel: 7,845 kg/m^3 (490 lb/ft^3), i.e., 76,980 N/m^3
- Aluminum: 2,642 kg/m^3 (165 lb/ft^3), i.e., 25,920 N/m^3
- Wood: 512 kg/m^3 (32 to 35 lb/ft^3), i.e, 5,030 to 5,500 N/m^3

- Concrete (foundations only): 2,400 kg/m^3 (150 lb/ft^3), i.e., 23,570 N/m^3
- Earth (foundations only): 1,760 kg/m^3 (110 lb/ft^3), i.e., 17,280 N/m^3

The dead load, strictly speaking, should be applied on the structure normal to the surface of the earth. For simplicity, this handbook permits the application of the dead load normal to the surface of the module provided the dead load to live load ratio is not greater than 0.4. The dead loads computed for the electrical and mechanical hardware may be assumed to be uniformly distributed over the surface of the array. However, if such hardware is so locally distributed and of such magnitude that to assume a uniform distribution would be incorrect and cause local overloading of the structural members, then the hardware should be added to the structural beam weight as a linear uniform weight (weight per unit distance). The weights of the structural beam may either be assumed to be uniformly distributed over the surface of the array, or be added to the combined load applied to the beams as discussed in Section 4.7.1.

4.3.2 Live Loads

Live loads are loads temporarily applied to the array structure not including wind load, earthquake load, or dead load.

To account for maintenance and service, a minimum live load should always be assumed and shall be 718 N/m^2 (15 lb/ft^2).

4.3.3 Wind Load

Wind load is the single most important load for the design of the array structure. It is usually several times larger than either the dead load or any other load.

The structures in this handbook have been designed under the following Wind Design provisions of the Uniform Building Code:

> Sec. 2311. (a) General. Buildings or structures shall be designed to withstand the minimum horizontal and uplift pressures set forth in Table No. 23-F* and this Section allowing for wind from any direction. The wind pressures set forth in Table No. 23-F* are minimum values and shall be adjusted by the Building Official for areas subjected to higher wind pressures. When the form factor, as determined by wind tunnel tests or other recognized methods, indicates vertical or horizontal loads of lesser or greater severity than those produced by the loads herein specified, the structure may be designed accordingly.

These provisions have been taken into account in the tables and figures in this handbook so that the designer need only find the wind pressure for the array location. To find the wind pressure in the United States for a 50-year mean recurrence interval, the Allowable Resultant Wind Pressure Map

* Figure 9 in this handbook.

and Wind Pressure-Height Table (Figure 9) from the UBC may be used. The designer first determines the allowable wind pressure for the array location from the map. Then the table should be consulted to adjust the allowable pressure for the height above the ground. The designer may optionally use Sec. 2311 (h) from the UBC for this calculation.

> (h) Miscellaneous Structures. Greenhouses, lath houses and agricultural buildings shall be designed for the horizontal wind pressures as set forth in Table No. 23-F*, except that, if the height zone is 10 feet or less, two-thirds of the first line of listed values may be used. The structures shall be designed to withstand an uplift wind pressure equal to three-fourths of the horizontal pressure.

Another source of wind load design criteria is the Timber Construction Manual published by the American Institute of Timber Construction. Excerpts from this manual are reproduced in Section 7.4 of this handbook. These excerpts may be used when:

- The terrain requires a more precise evaluation of wind loads.
- A mean recurrence interval other than fifty years is required.
- The recommended wind velocity is known but not the recommended wind load. In this case, the following formula from the Timber Construction Manual may be used:

$$L = 0.00256 \times v^2 \times (H/30)^{2/x} \times G_F$$

where

L	wind pressure
v	wind velocity in mph
H	height above grade (in feet) of pressure being computed
x	exponent depending upon general site exposure conditions
G_F	gust response which is a function of terrain

For wind load criteria outside the United States, the designer should consult local codes and requirements. In lieu of this information, the United States Army and Air Force Manual TM5-809-1 AFM 88-3 should be consulted. Wind, snow and frost data from this manual are reproduced in Section 7.5 of this handbook.

4.3.4 Seismic Load

Seismic loads are those loads induced in a structure due to ground motion caused by earthquakes and volcanic action. Since seismic loads are related to the dead load weight of the structure (loads are introduced into the structure due to the inertia of the structure when the ground moves), and since these structures are relatively light, only small seismic forces are

* Figure 9 in this handbook.

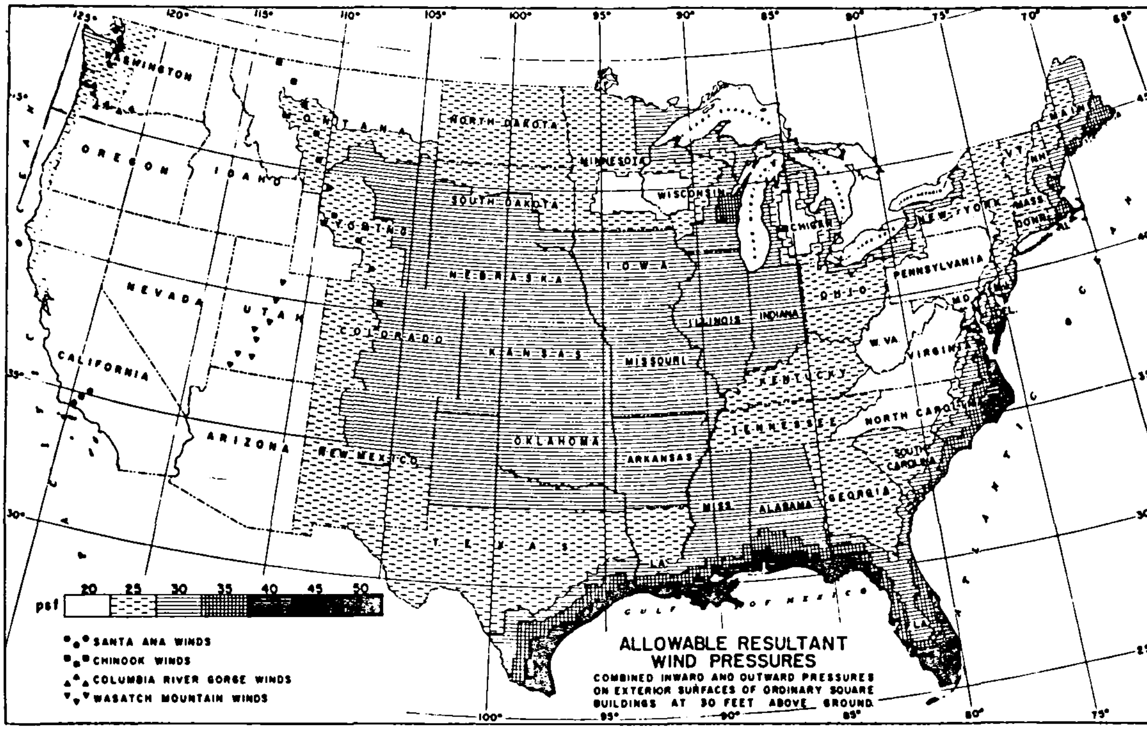


FIGURE NO. 4

HEIGHT ZONES (in feet)	WIND-PRESSURE-MAP AREAS (pounds per square foot)						
	20	25	30	35	40	45	50
Less than 30	15	20	25	25	30	35	40
30 to 49	20	25	30	35	40	45	50
50 to 99	25	30	40	45	50	55	60
100 to 499	30	40	45	55	60	70	75
500 to 1199	35	45	55	60	70	80	90
1200 and over	40	50	60	70	80	90	100

Figure 9. Allowable Resultant Wind Pressures (UBC)*

* Reproduced from the 1976 edition of the Uniform Building Code, Copyright 1976, with permission of the publisher, the International Conference of Building Officials.

introduced. By comparing seismic forces with the wind forces, the designer can see that in almost all cases the wind load will prevail.

The basic formula for determining seismic forces is:

$$V = (Z \times I \times K \times C \times S) \times W$$

where

V	total lateral seismic force
W	weight per volume of modules and array structure. (This may be estimated from the values given in Section 4.3.1 if the overall array size is known.)
Z,I,K,C,S	variables whose values depend on the type of structure and its location

A simplified conservative formula for seismic load may be used which will be satisfactory for most locations. This formula is:

$$V = 0.24\ W$$

If a more accurate method is preferred, the designer should use the UBC. This method is reproduced in Section 7.6 of this handbook.

4.3.5 Snow Loads

Snow loads are those loads on the array structure due to accumulation of snow fall on the surface of the array. Snow drifting onto the array is not a situation addressed by the codes, but an analysis of code provisions for snow loads on roofs indicates that a similar approach may be used for photovoltaic array structures.

Snow loads on roofs are determined by the measured snow load on the ground for a specific mean recurrence interval. For the United States for a 50 year mean recurrence interval, the map in Figure 10 from the AITC Timber Construction Manual may be used to determine the ground snow load. Additional material from this manual is reproduced in Section 7.7 of this handbook for use when:

- A mean recurrence interval other than 50 years is required.
- Figure 10 provides no ground snow data for the array location.
- The array is located next to a large projection from the ground such as a building or natural bluff.

See also Section 7.5 of this handbook for snow load data at various locations inside and outside the U.S.

Normally to determine the basic snow load on a roof, the ground snow load is multiplied by a coefficient of 0.8. This coefficient can be used when an array structure is mounted a significant distance above the ground, which is the case for the pole-mounted structure. Triangular frames discussed in this handbook have a basic snow load determined by using the ground snow load without the coefficient, since these structures are to be mounted near the ground surface.

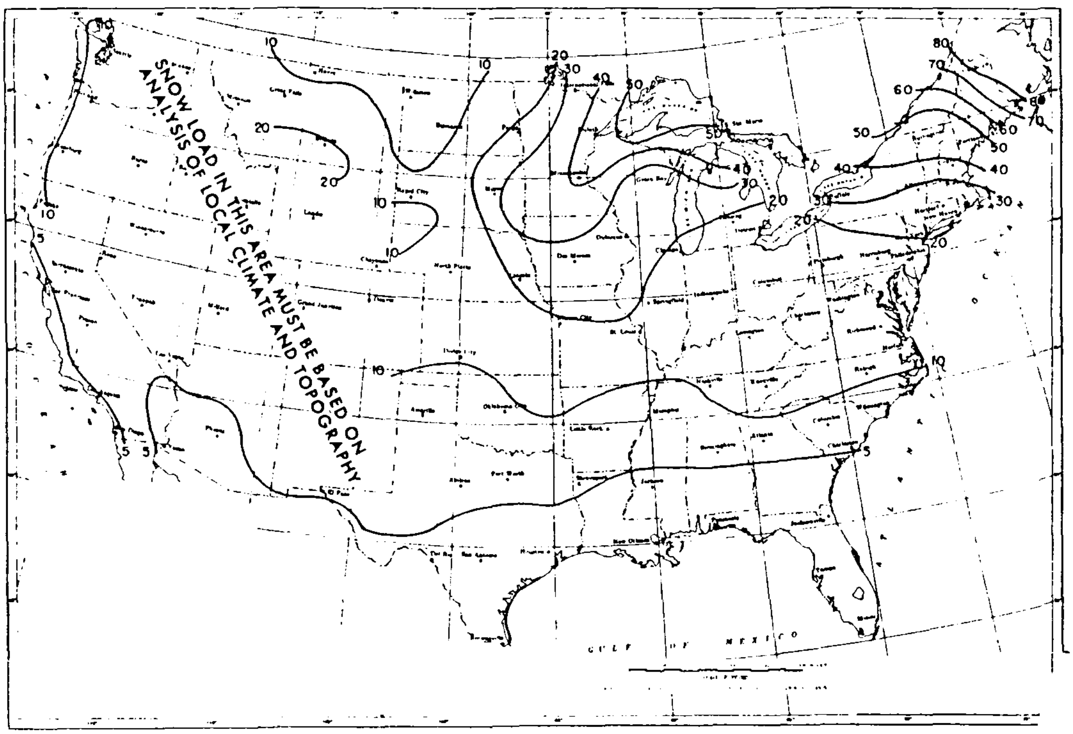


Figure 10. Snow Load in Pound Force per Square Foot on the Ground, 50-year Mean Recurrence Interval

Snow loads in excess of 718 N/m^2 (15 lb/ft^2) may be reduced if the angle of inclination of the module is greater than 20^o. The formula for the reduced snow load is (See Section 7.7):

$$S_R = 0.8 \times S - (0.025S - 0.5) \times (\theta - 20)$$

where

S_R	reduced snow load
S	snow load from ground data
θ	angle of inclination of module in degrees

4.3.6 Ice Loads

Ice loads are those loads that result from the accumulation of ice on the structure. This state can occur for a variety of reasons, but the two most common are freezing rain, and the thawing and refreezing of fallen snow. In general, for array structures this load is not greater than the other live loads. Local authorities should be consulted about special provisions for ice loads if there is a concern about such loads.

4.3.7 Thermal Loads

Thermal loads are those loads caused by the effects of temperature and temperature change on the structure. These effects include material fatigue and differential expansion, each of which could cause the structure to fail. However, for the type of simple structures discussed in this handbook, assuming good design practice is followed and the weather conditions are not extreme, thermal loads are not a significant problem.

4.3.8 Miscellaneous Loads

Miscellaneous loads are those loads which usually do not affect the majority of structures, but could affect structures under special conditions. These loads are of such a unique nature that special codes are written for them: blast proof design, tornado design, special use or industrial design, and drifting sand design. The designers of photovoltaic arrays should ascertain if the structure will be subjected to additional loads other than those discussed above and design accordingly.

4.3.9 Combined Loads

Loads do not act on an array structure independently; certain loads will combine to stress the structure. A structure designed to withstand the sum of all the possible loads will be an oversized, uneconomical structure. The reason for this is that all loads described in Sections 4.3.1 through 4.3.8 inclusive are maximum loads and it is improbable that all maximum loads will occur at the same time on the array. So the design must use some method of "weighting" the different maximum loads to determine a reasonable combined design load.

One method is to calculate the combined load as the dead load plus the largest of all the other loads, which is usually the wind load.

A second method, based on load combination probabilities, is given in the BOCA Code. In this approach, the combined load (P) is determined to be the largest of the following eight combinations:

- dead load
- dead + live loads
- dead + (wind or seismic) loads
- dead + thermal loads
- (dead + live + (wind or seismic) loads) times 0.75
- (dead + live + thermal loads) times 0.75
- (dead + (wind or seismic) + thermal loads) times 0.75
- (dead + live + (wind or seismic) + thermal loads) times 0.66

Either method may be used to determine a reasonable design combined load.

4.4 Environmental Factors

In addition to the load factors previously disclosed there are additional environmental factors the photovoltaic array structural designer should consider. These factors include among others: water, temperature, pollution and soil heaving.

4.4.1 Water

Humidity is atmospheric water vapor content while rain is precipitation of liquid water. Both can significantly affect structures.

The obvious effect humidity and rain may have on structures is oxidation (commonly referred to as rust for ferrous metals and alloys). Oxidation occurs for both steel and aluminum. Fortunately, aluminum's oxide coat protects it from further oxidation. In fact, due to the protective nature of aluminum oxide many designers will specify that the aluminum be anodized. This is done by an anodic process in a suitable electrolyte such as chromic acid or sulfuric acid solution. Caution should be used when specifying anodizing aluminum since the oxide must be removed by some means prior to soldering, brazing or welding. The designer should refer to the American Welding Society specifications or the Aluminum Association's specification for the proper requirements.

Steel also oxidizes, but unlike aluminum the oxide does not protect the metal. Steel must be continually maintained by painting to prevent it from being damaged by oxidation.

However, special alloy steels are commercially available which have better than four times the corrosion resistance of carbon steel. The ASTM designations for such steels are ASTM A242 and ASTM A588 and they may be used in the steel structures found in Section 4.6.2.2 in lieu of ASTM A36 steel.

These weathering steels may be used without painting and are especially effective where infrequent maintenance is desired. The designer should be aware that these weathering steels are higher strength steels than the A36 steel and consequently lighter steel sections than those shown in the tables may be used. As an option, one might reduce the design live load by 10 percent to account for the higher strength steels. However, the loads should not be reduced beyond 10 percent because the relationship between strength of one steel to another is not strictly linear. In lieu of an exact analysis and design by a professional structural engineer the sizes as shown in the tables should be used. Weathering steel has one bad side effect: leaching. Consequently, it should be avoided where staining and streaking is undesirable. Steel may also be galvanized where long maintenance-free periods are desired. Galvanized steel should be touched up in the field with a galvanizing paint after installation. Recent experience with pricing of weathering steel, galvanized A36 steel, and aluminum indicates that weathering steel is generally the least costly.

Wood is also severely affected by humidity and rain. In fact, a sizable number of wood structures have collapsed due to rain and high humidity. These structures have ranged from barn roof trusses caught in the rain during construction to huge industrial, redwood timber cooling towers. The cause of the collapses was the increased moisture content of the wood. Wood is stress rated at a specified moisture content. As the moisture content of the wood increases the allowable stress rating of the wood goes down. For example, should the moisture content of the wood rise to 19 percent the allowable stress decreases about 10 percent. A sizable increase in moisture content could collapse an array structure of wood. The increase in atmospheric moisture content either in the form of humidity or rain would have to be of a prolonged nature to weaken a wood structure. Also, the wind load would have to increase to the design load. Prolonged atmospheric moisture increase would have to exist for weeks, or even months, without the opportunity for the wood to dry out. Tropical rain forests and areas with prolonged monsoon seasons are likely candidates to harm a wood structure. One must note that a properly painted wood structure would not be as susceptible to moisture content increase. Preservative pressure treatment, however, offers no benefits in terms of preventing moisture content increases.

Concrete is also affected by rain and humidity. Rain on concrete prior to its setting up (hydration) can weaken the concrete since concrete's strength is a function of its cement to water ratio. In general, excess water in a concrete mix lowers the strength of the concrete. High humidity will not cause adverse effects in solid concrete, but low humidity and high temperatures can cause deleterious effects such as spalling and cracking.

4.4.2 Temperature

Change in ambient temperature which causes expansion and contraction in a structure does induce stress (force per unit area) in the structure as does differential settlement of the foundation of a structure; and, both should be considered in the design of a structure. However, for the structures in this handbook stresses due to differential settlement and thermal expansion are insignificant. Thermal loads may become significant in structural systems over 50 meters (164 ft) long; while differential settlement will become significant for structures in this handbook when it approaches a quarter inch. Additionally, as discussed in Section 3.2.2, differential thermal expansion and contraction between the module and the structure appear to be rather insignificant as a rule. This tends to be true in part because the coefficient of thermal expansion of the various materials for structures and modules are of the same relative magnitude, and because the modules tend to be rather small while the structural elements tend to be rather large; so, though the modules may expand or contract relative to the structure, the effect is not cumulative over the length of the structure due to the segmental nature of the modules and therefore, the induced stress is small.

Extreme cold as that experienced at the earth's poles will require special consideration since most structural metals will become brittle. Welding, as well, may require special techniques.

Foundations on tundra and permafrost are complex engineering problems and are beyond the scope of this handbook. Tundra is the meager vegetation between the limit of trees and that of perennial snow and ice on mountains or about the earth's poles. Permafrost is perennially frozen ground. A foundation built through tundra into permafrost has a significant consequence beyond the fact that tundra is destroyed. The foundation which may have been warmed by conduction from the photovoltaic modules through the structure to the foundation may thaw the Permafrost. Consequently the foundation will settle. This settlement could be severe and even involve rotation of the foundation. A geotechnic engineer specializing in foundations on Permafrost should be consulted when the array designer is faced with this problem.

4.4.3 Pollution

Saltwater environments create a damaging atmosphere for metal structures. Again, oxidation is the critical process. Aluminum should be anodized and ASTM A36 steel should be painted. Similar precautions should be taken in industrial environments where corrosive chemicals precipitate out of the atmosphere onto the structure.

Pollution will also have an effect on the ability of the cells to receive radiation. In addition to the effect of pollutants in the air permanently reducing the radiation from the sun to the module superstrate, the pollution will also reduce the radiation from the module superstrate to the cells in two ways. First, chemicals from the air will adhere to the superstrate surface, blocking sunlight to the cells. Secondly, the chemicals may cause a change in the material or chemical configuration of the module superstrate. While the first effect is a temporary effect which may be removed by rain or regular cleaning, the second effect, especially for silicone rubber superstrates, will cause a "cloudiness" in the superstrate which will reduce the efficiency of the module. If the modules do not receive regular cleaning, the first effect may also become permanent.

For these reasons, it is necessary to determine the type and amount of pollution at the site before module and structural material selection are completed. It may be necessary to increase the initial cost of the system, by using more expensive modules and structural materials, if the site is a highly polluted area and/or maintenance will be infrequent.

4.4.4 Soils Heaving

Water freezing in soil causes the soil to heave. Therefore, if a foundation is placed such that its bottom surface is located above the frost line then the foundation will move. Movement of a foundation alone will not be too serious for a pole-mounted structure provided that the foundation does not rotate and the foundation returns to its original elevation in the spring when the ground thaws. However, structures other than pole-mounted structures are severely affected by such movement for the foundations of a complex structure would move differentially thus causing large internal stresses in the structure not accounted for in the design. Therefore, it is important to locate the bearing surfaces of foundations below the maximum local frost penetration. This is covered in more detail in Section 4.7.1.5.

4.5 Generic Structures

4.5.1 Types of Generic Structures

A generic structure is a schematic design for supporting an applied load. There are a number of categories of generic structures suitable for photovoltaic arrays. The structures included in this handbook are the dedicated (permanent) triangular frame, portable triangular frame, and pole-mounted structure. Figure 11 illustrates the different types of dedicated triangular framing systems covered by this handbook, including size limits. Figure 12 illustrates the pole-mounted system. Figures 13A and 13B show a detailed side view of the one- and two-tier dedicated triangular structures. Figures 13C and 13D illustrate two sample types of portable triangular structures.

The triangular framing system is currently the most popular structure for photovoltaic arrays. It has the advantages of simplicity of design, a minimum number of required connections, and ease of construction, installation, and maintenance. The main disadvantage is that this system may prove expensive depending on availability and requirements for specific construction materials.

The portable system is useful for the armed forces and others for whom temporary power is required. Since the "temporary" period may extend anywhere from a few hours to a few days, the structure must be capable of supporting itself. The type of portable system used in this handbook utilizes the triangular framing structure without the use of the permanent concrete supports. There are several methods available for anchoring the portable structure to the ground. Among them are: (1) attaching the ends of the columns to small T (tee) pieces (Figure 13C) which are then pinned to the ground or covered with weights such as sandbags; and (2) connecting frame support points A and B together with a spreader bar (Figure 13D) and applying weight to the spreader bar. When determining the weight to be applied to resist overturning, the amount of time the structure will be in one place should be taken into consideration. The weight must be equal to the uplift force calculated for the worst case condition. The uplift force is greatest when the wind blows from behind the array, therefore the design wind load should be increased by 25 percent (UBC Section 2311 (c)). The beam reactions must be determined and the equations of equilibrium

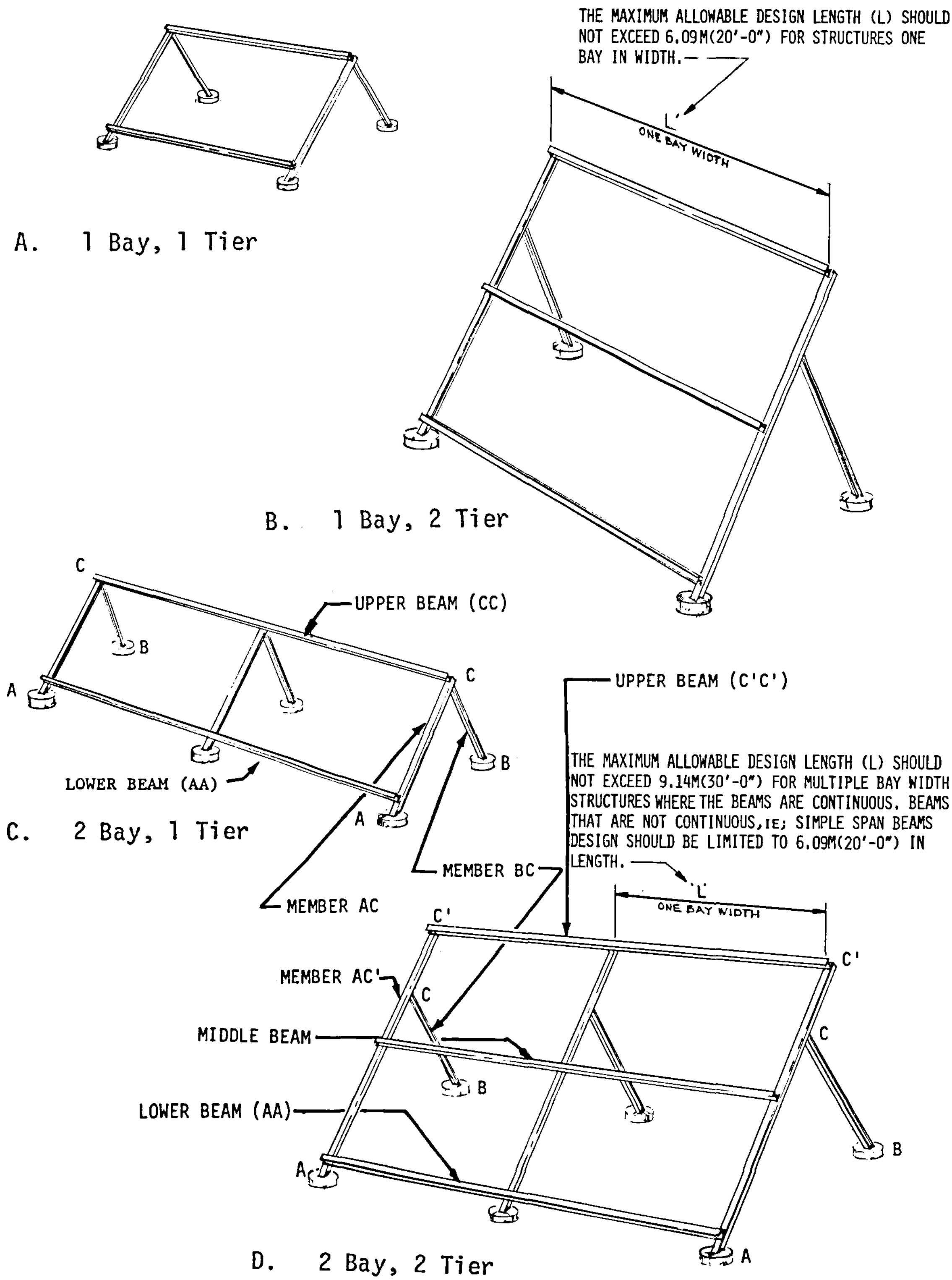


Figure 11. **Dedicated Triangular Frame Systems and Size Limits**

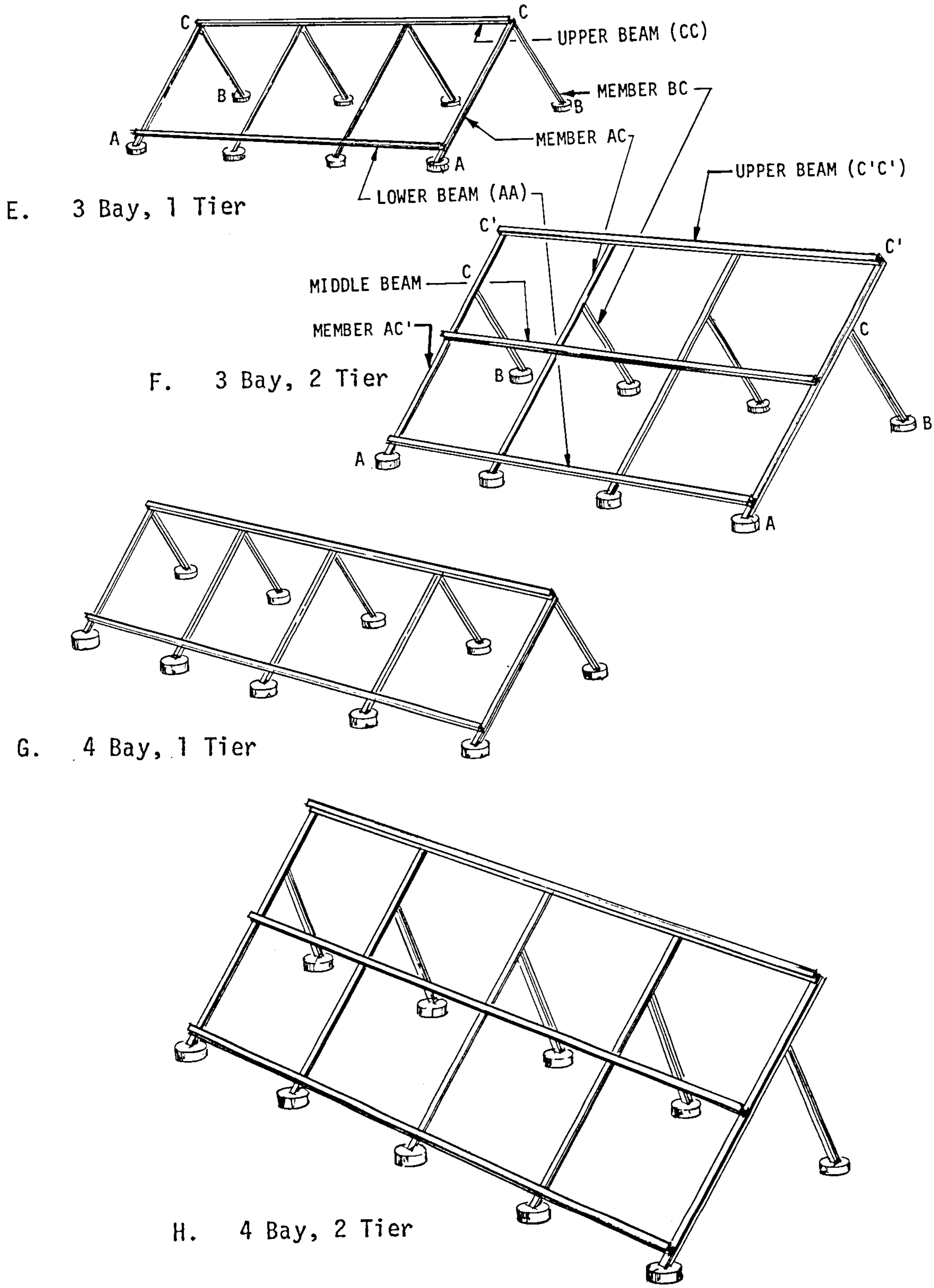


Figure 11. Concluded

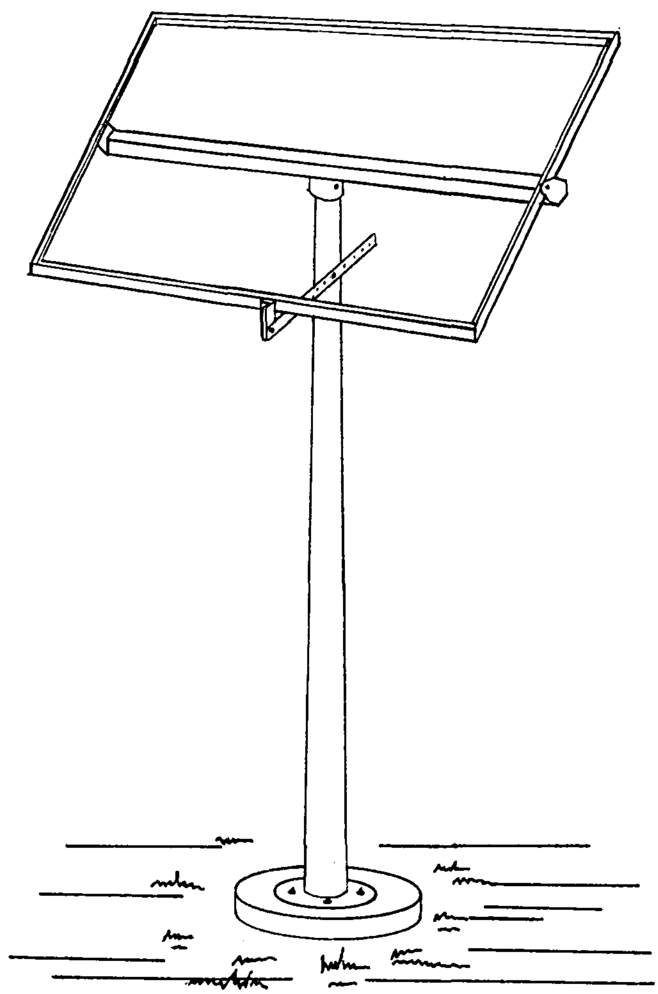
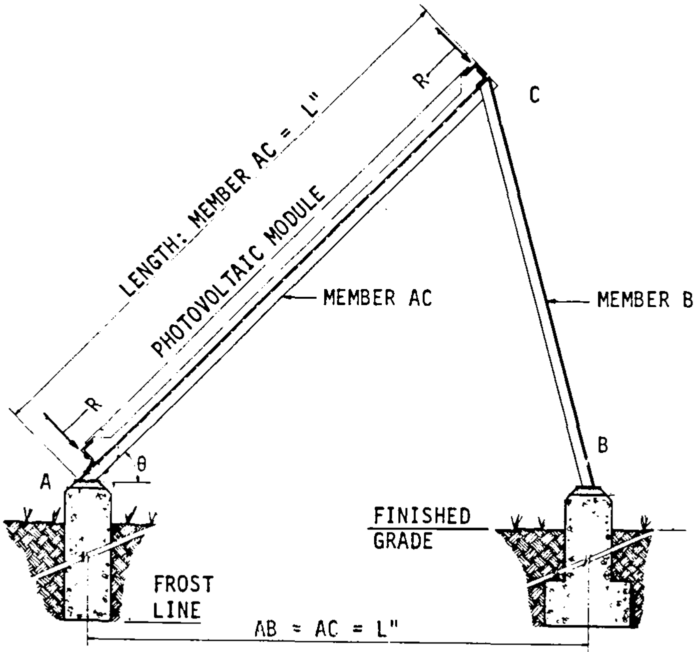


Figure 12. Pole-Mounted System

Units: CM Unless Otherwise Noted



A. One-Tier Structure

B. Two-Tier Structure

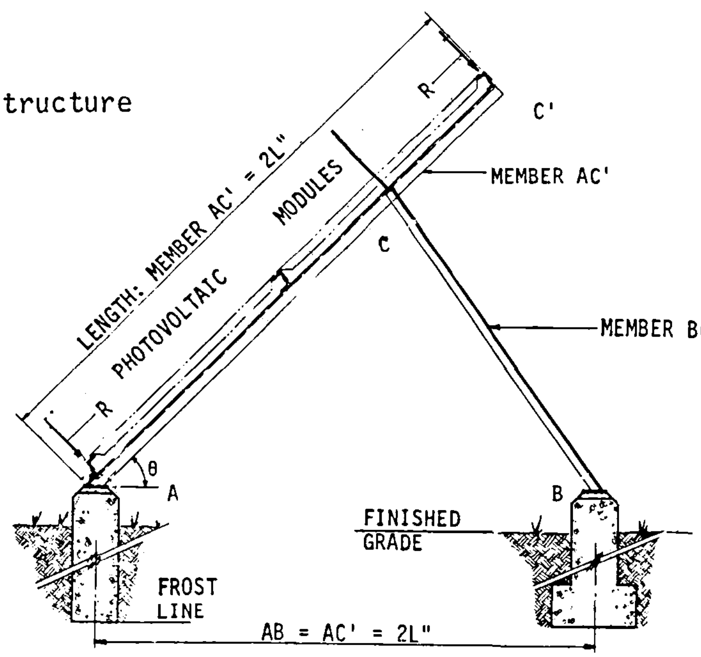
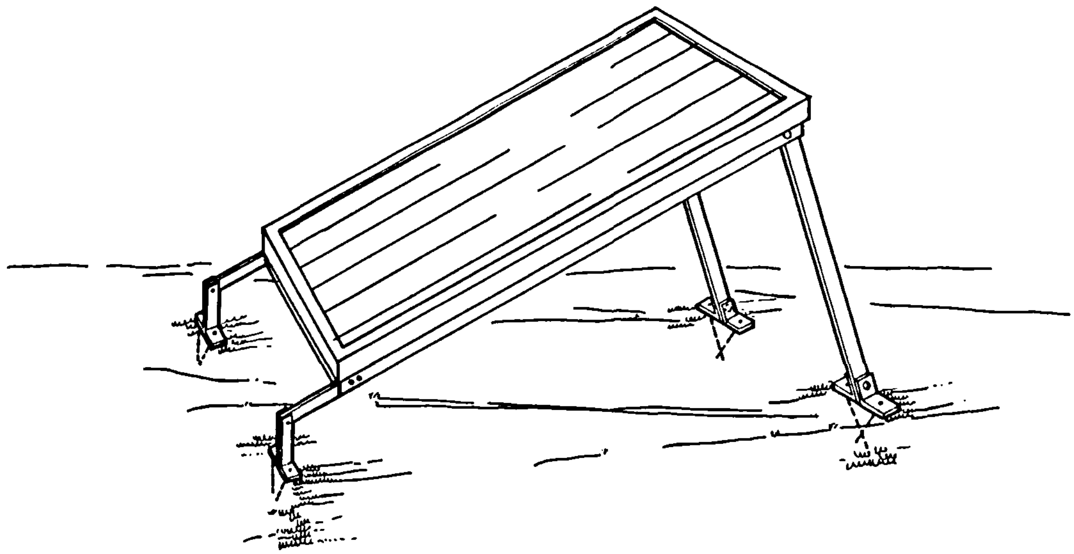
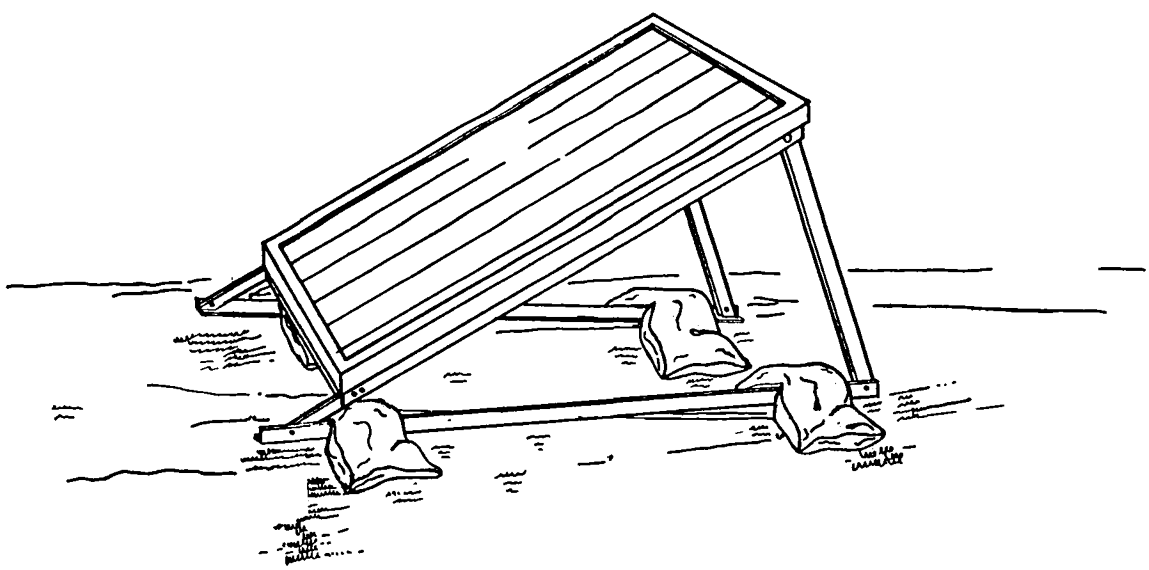


Figure 13. Generic Structural Systems



C. Type 1 Portable Structure



D. Type 2 Portable Structure

Figure 13. Concluded

resolved. The weights must be able to resist the forces described in Section 4.5.2.

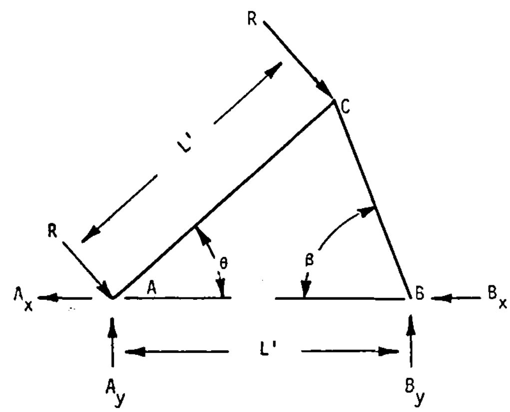
Finally, the pole-mounted system is a simple and inexpensive structure for smaller arrays. Site work and the number of members required are reduced considerably over the dedicated triangular frame systems. It is not feasible for large arrays or at large distances above the ground, i.e., greater than 4 m (12 ft) which would require significantly larger members and poles than covered in this handbook to resist the combined loads.

It should be noted that either the dedicated triangular frame structures or the portable frame structures might also be roof-mounted by bolting frame support points A and B to the roof, providing the roof is sturdy enough to support the forces exerted on the array.

4.5.2 Design Methodology

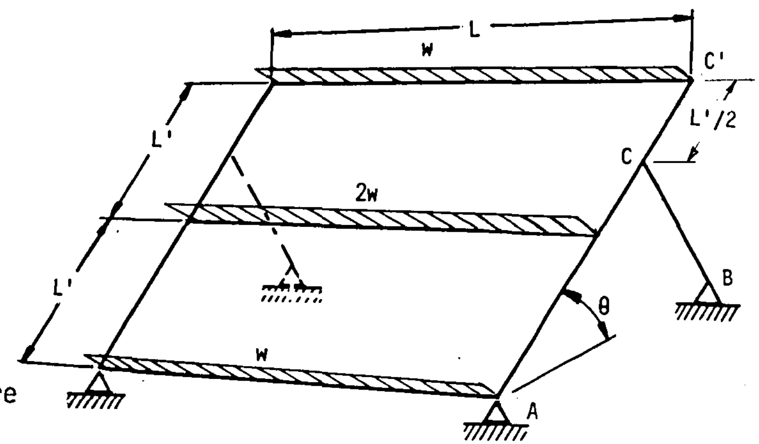
This section discusses the design methodology used for designing triangular frame array structures. The design methodology discussed in this section is the basis for development of the design algorithms and tables presented in Section 4.7.

The following diagram and Figure 14B illustrate the array loading for the one-tier structure:

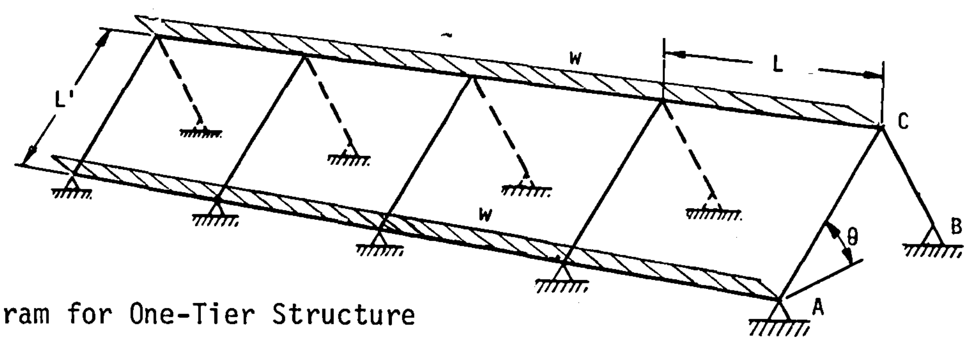


Having considered the loads applied to the structure, and having determined a generic structural system, the designer proceeds to the analysis of the system and then to the selection of materials whose shape, size, and mechanical properties are sufficient to resist the externally applied loads. This requires application of Newton's third law which may be simply stated as follows: The forces of action and reaction between contacting bodies at rest are equal in magnitude, opposite in direction and collinear. This is the basis of static analysis for planar structures. Practically speaking, this means that the sum of forces in both the horizontal and vertical direction are equal to zero and that the moment about any point on the structure must be equal to zero. Moment is the tendency of a force applied to a structure to rotate the structure about any axis which does not intersect the line of action of the force. The static analysis of a one-tier triangular structure is shown above; where R = the applied force and A, B, C are the boundaries of the structure. The equations of equilibrium are: $\sum F_x = 0, \sum R_y = 0, \sum M_z = 0$.

θ = TILT ANGLE
w = LOAD PER LINEAR FOOT ON THE BEAM
L = ONE BAY WIDTH
L'= NOMINAL DISTANCE BETWEEN BEAMS



A. Loading Diagram for Two-Tier Structure



B. Loading Diagram for One-Tier Structure

Figure 14. Beam Loading Diagrams

Since R is known and all of the geometry is known, the unknowns that must be determined are A_x, A_y, B_x, B_y. These are forces at support points A and B (the direction of action is indicated by the subscript) required to resist the applied force R if the structure is to remain stable. Once these forces are determined, then members AC and BC can be designed to carry the forces; the supports (usually concrete foundations in soil) to resist the forces at A and B must also be designed.

Resolutions of the equations of equilibrium for the one-tier structure are as follows:

$$\sum M_{@A} = 0 = L'R - L'B_y$$

$$B_y = R \tag{1}$$

$$\sum F_y = 0 = B_y + A_y - 2R\cos\theta$$

$$A_y = 2R\cos\theta - R \tag{2}$$

$$\tan\beta = B_y/B_x$$

where

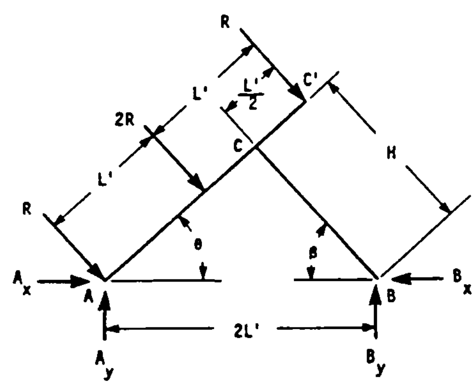
β internal angle at support **B**, $(180-\theta)/2$

$$B_x = B_y/\tan\beta \qquad B_x = R/\tan\beta \tag{3}$$

$$\sum F_x = 0 = 2R\sin\theta - A_x - B_x$$

$$A_x = 2R\sin\theta - R/\tan\beta \tag{4}$$

The following diagram and Figure 14A illustrate the array loading for the two-tier structure:



Resolution of tne equations of equilibrium for the two-tier structure are as follows:

$$\sum M_{@A} = 0 = 2RL' - 2L'B_y$$

$$B_y = 2R \tag{1}$$

$$\sum F_y = 0 = B_y + A_y - 4R\cos\theta$$

$$A_y = 4R \cos \theta - 2R \qquad (2)$$

$$\tan \beta = B_y/B_x$$

where

β internal angle at support B

$$\beta = \text{arc} \cos \left(\frac{H^2 + 1.75\, L'^2}{4\, HL'}\right), H = \sqrt{6.25\, L'^2 - 6\, L'^2 \cos \theta}$$

$$B_x = B_y/\tan \beta \qquad B_x = 2R/\tan \beta \qquad (3)$$

$$\sum F_x = 0 = 4R \sin \theta + A_x - B_x$$

$$A_x = 4R \sin \theta - 2R/\tan \beta \qquad (4)$$

In the system chosen, the one- and two-tier structure is a frame. The modules are attached to the beams that span the triangular frames. These beams act as loading points on the frame.

In the one-tier structure, member AC and member BC act as columns. The direction of the wind determines whether the member is in tension or compression. The axial stress in the column is determined by the reaction transmitted from the beam to the frame. As shown previously the resolution of the equations of equilibrium results in the determination of B_x, B_y, A_x, and A_y. The axial force in member BC is $\sqrt{B_x^2 + B_y^2}$. From these forces the actual axial stress can be computed: $f_a = F_b/A_c$ where F_b is the axial force and A_c is the cross sectional area of the member. The members are designed such that $F_a > f_a$ where F_a is the allowable axial stress (dependent on the material). Both member BC and member AC should be designed for compression because buckling is more critical than tensile strength. Member AC should be designed to withstand 1.25 times its actual axial force (see UBC Section 2311(c)). This is due to the observation that member AC is in compression when the wind blows from behind the modules and uplift occurs.

In the two-tier structure, member BC acts as a column and member AC' acts as a beam-column. Since member AC' is subject to both bending and axial stresses, it must be designed in accordance with an interaction formula as dictated by the material used. For example the interaction formula for steel reduces to $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$ when $\frac{f_a}{F_a} \leq 0.15$

where

f_a actual axial stress
F_a allowable axial stress

f_b actual bending stress
F_b allowable bending stress (AISC Section 1.0.1)

The actual axial stress is found in the same manner as shown for the one-tier structure. The actual bending stress is M/S where M is the maximum moment developed in the member and S is the section modulus of the member.

The beams that span the triangular frames support the modules which subject the beams to a uniform load. This situation was shown in Figure 14. For a simple span beam the maximum moment $M = WL^2/8$ where W is the uniform load and L is the span length. The beam is designed such that $f_b \leq F_b$ (again $f_b = M/S$). The allowable stress in all cases may be increased by 25 percent since the major load is wind load (See UBC Section 2303 (d)).

The foundation is designed to resist overturning of the structure in addition to horizontal sliding. According to the UBC Section 2311 (i) the dead load resisting moment (M_R) must be 1-1/2 times the overturning moment (M_O) calculated from wind pressure. The dead load resisting moment is calculated from the weight of the earth over the footings of the structure.

4.6 Structural Materials Selection

4.6.1 Basic Materials

The basic materials considered in this handbook are structural steel, aluminum, and cold-formed steel. Wood is also considered for pole-mounted structures. These materials were chosen because they are the most widely used structural materials. The primary criteria for material selection are: availability and cost, maintenance, and lifetime. The major criterion, especially for small systems, is usually the initial cost of the system. In the case of extreme environments, however, the other criteria may play an important role in the selection of structural materials. Before selecting the materials to be used the designer should review the discussion on environmental effects on structural materials given in Section 4.4 of this handbook.

A short description of the nature of the materials covered in this handbook is given below.

A. Structural Steel

The amount of carbon present in steel affects the physical properties. It may vary from 0 to 1-1/2 percent. Increasing the amount of carbon increases the strength, hardness, and brittleness of steel but decreases its ductility. Structural steels are identified by an ASTM (American Society for Testing and Materials) designation number that specifies the steel. The most commonly used structural steel is ASTM A36 steel, a carbon steel.

B. Aluminum

Aluminum alloy structural shapes are produced either by extruding or rolling. Some aluminum alloys are heat treatable; others are not.

A widely used heat-treatable wrought aluminum alloy is 6061-T6. The number 6061 identifies the alloy composition; four digits distinguishes it is a wrought rather than a cast alloy. The "-T" shows that the metal has been heat treated; the final "6" indicates the type of heat treatment.

C. Cold-Formed Steel

Cold-formed steel structural members are produced from steel sheet, strip, plate or bar stock. The forming is done in a roll former or press brake. The specified minimum yield point is the primary criterion for strength under static loading. Structural sections of cold-formed steel have not been standardized. Some fabricators have developed their own structural sections.

Cold working has an effect on the mechanical properties of ductile metals. In general the yield strength of the steel is raised to a large degree by cold working, the tensile strength to a smaller degree, and the ductility is reduced. Since the effects of cold working are still obscure, the allowable design stresses are based on the properties of the flat material before forming.

D. Wood

The terms wood, lumber, and timber are often used interchangeably but they do have distinct meanings. Wood is the substance forming the trunk and branches of trees. Lumber is the product of the sawmill. Timber is lumber 13 cm (5 in.) thick or larger. Lumber is provided by two classes of trees: softwood (pines, firs, spruces) and hardwood (maple, oak, sycamore). Softwood is used in structural applications such as framing. Hardwood is generally used in flooring, paneling and furniture.

Wood is seasoned to prepare it for construction. Seasoning the wood involves air drying or kiln drying. The results of seasoning are reduced moisture content and an increase in strength and resistance to decay. The moisture content affects several mechanical and physical properties of wood. A reduction of the moisture content increases resistance to fungi attack and improves structural ability. The moisture content (expressed as a percentage) is the weight of the moisture in a sample of wood divided by the oven dry weight. Lumber decay is caused by fungi, which feed on the cell walls. To develop the fungi require warmth, air, and moisture. When lumber is used where moisture is present, seasoning loses its effectiveness. To check decay substances must be introduced into the lumber that will poison the fungi. These substances are called wood preservatives. The preservatives are applied in such a way that the preservative is left impregnated in the wood. There are many methods of application: spraying, dipping, open-tank process, and pressure. This reference recommends pressure treatment because it is the most effective.

4.6.2 Alternate Systems and Materials

In addition to the systems and materials covered in this handbook a number of alternate systems and materials are commercially available which may prove cost-effective for small arrays. These include the following:

- Space Frames
- Stock Framing Systems
- Stock Perforated and Slotted Shapes
- Plastic Shapes
- Plastic Pipe and Fittings

A further discussion of these systems and materials is given in Appendix A of this handbook.

4.7 Structural Design

4.7.1 Structural Design of Triangular Framing Systems

This section is divided into seven parts: the structural design procedure; the properties and specifications sections for structural steel, aluminum, and cold-formed steel; the foundation design specifications and information section; the triangular framing system design figures; and the triangular framing system design tables.

4.7.1.1 Structural Design Procedures

A. Determine W: uniform load applied to beams. (See Figure 14.)

(a) Top and bottom beams

$$W_{TB} = P \times L'/2$$

where

W_{TB}	uniform load applied to top and bottom beams
P	combined load determined in Section 4.3.9
L'	height dimension of one module, i.e., distance between beams

(b) Middle beam (for two-tier structures)

$$W_m = P \times L'$$

where

W_m	uniform load applied to middle beam

B. Determine M: the maximum moment developed in the beams.

(a) Simple span and two continuous span beams: top and bottom beams

$$M_{TB} = 1.25\ W_{TB}L^2/8$$

---

* All sizes are nominal American shape sizes in inches. Conversions to metric units are given in Section 6.2. Metric shape sizes may not correspond exactly to American shape sizes.

where

M_{TB}	maximum moment developed in top and bottom beams
L	span length between support frames evenly spaced
1.25	factor for wind load reversal

(b) Three continuous span beams: top and bottom beams

$$M_{TB} = 1.25\ W_{TB}L^2/10$$

(c) Four continuous span beams: top and bottom beams

$$M_{TB} = 1.25\ W_{TB}L^2 \times 0.1071$$

(d) Simple span, and two, three, and four continuous span beams: middle beam (two-tier structure)

$$M_m = M_{TB} \times 2$$

where

M_m	maximum moment developed in middle beam

C. <u>Determine R</u>: The maximum reaction for the top and bottom beams (See Figures 13A and 13B)

(a) Simple span beams

$$R_{TB} = W_{TB} \times L/2$$

(b) Two continuous span beams

$$R_{TB} = W_{TB} \times L \times 1.25$$

(c) Three continuous span beams

$$R_{TB} = W_{TB} \times L \times 1.10$$

(d) Four continuous span beams

$$R_{TB} = W_{TB} \times L \times 1.143$$

D. Determine sections to be used for top and bottom beams.

(a) Proceed to:

(1) Material, construction, shape and splicing notes at the beginning of Section 4.7.1.2, then to Table III for <u>structural steel</u>

(2) Material, construction, shape and splicing notes at the beginning of Section 4.7.1.3, then to Table IX for <u>aluminum</u>

(3) Material, construction, shape and splicing notes at the beginning of Section 4.7.1.4, then to Table XII for <u>cold-formed steel</u>

(b) Find the member in the table which has the smallest value of M_{max} which is larger than M_{TB}.

(c) Find the weight/length for this member from the table.

(d) Divide the weight/length by L to get weight/length^2.

(e) Correct the value for P which was calculated in Section 4.3.9.

(f) Go back to step A and recalculate all equations using the new value of P determined above for steps A through C; determine sections to be used for top and bottom beams by the method described in step D, above.

(g) If the member determined in step (f) is the same as the one previously determined, then this is the correct member size for the top and bottom beams. If they are not the same, repeat step (f).

E. Determine section to be used for middle beam (two-tier structure)

(a) Notes for each material:

(1) Structural Steel
The middle beam must be a T (tee) or wide flange. It must also have the same depth as the top and bottom beams or mounted so its top surface is flush with the top surface of the top and bottom beams. If the beam chosen is unequal leg angles back-to-back forming a T (tee), the short legs should be attached to the modules. Also, if the beam is excessively long, it will require field splicing.

(2) Aluminum
See steel notes above.

(3) Cold-Formed Steel
The middle beam must be two channels back to back. It must also have the same depth as the top and bottom beams. If the beam is excessively long, it will require field splicing. See Tables XVI and XVII for schedules of different cold-formed steel shapes.

(b) Follow the procedure given in step D above, substituting M_m in place of M_{TB}.

F. Determine lengths: for members AC (AC' for two-tier structures) and BC (See Figures 11 and 16).

(a) For one-tier structure:
The length (L'') of member AC equals the separation distance between the foundation supports. Given the tilt angle, θ, the length of member BC is

$$BC = \sqrt{2L''^2 - 2L''^2 \cos \theta}$$

(b) For two-tier structure:
The length ($2L''$) of member AC' equals the separation distance between

the foundation supports. The connection of member AC' to member BC, however, occurs at 1.5 L" from point A. For this case the length of member BC is

$$BC = \sqrt{6.25L''^2 - 6L''^2 \cos \theta}$$

G. Determine sections to be used for members AC (AC') and BC (See Figures 11 and 16)

(a) Proceed to:

(1) Table IV for structural steel one-tier
(2) Table V for structural steel two-tier
(3) Table X for aluminum one-tier
(4) Table XI for aluminum two-tier
(5) Table XIV for cold-formed steel one-tier
(6) Table XV for cold-formed steel two-tier

(b) Find the member in the member AC (AC') section of the table which has the smallest value for R which is larger than R_{TB}. This will serve as member AC (AC'). Repeat for member BC.

Note: The sections listed in the tables are sized for the worst case load on the interior frame(s) when using the value for R_{TB} calculated in step C, above, as the criterion for member selection.

H. Determine connection details:
The connection details may be developed from Figure 16 and the following:

(a) Figures 17 to 22 for structural steel.

(b) Notes on connections at the beginning of Section 4.7.1.3 and Figure 31 for aluminum.

Note: Aluminum sections are similar to structural steel, so the designer should refer to the structural steel connection figures.

(c) Figures 32, 33, 34, and 35 for cold-formed steel.

I. Determine bracing details:
The bracing types considered in this handbook are cross bracing (single span structures only) and diagonal bracing. Bracing members consist of rods (tension members) and compression members. Bracing details may be developed from Figures 23 to 30 and Table VIII.

Note: Bracing is generally not needed for structures with low tilt angles ($\leq 15^0$).

J. Determine foundation required:

(a) Read the General Notes for Section 4.7.1.5

(b) Foundation for one-tier and two-tier structures:

(1) Turn to Tables XXIII for one-tier structures and XXIV for two-tier structures.

(2) Find the support point A and support point B rows corresponding to the elevation angle θ measured from the horizontal of the array.

(3) Find the column in these rows corresponding to the smallest value of R which is larger than R_{TB}. The values at the intersection of the columns and rows are the appropriate foundation types for support point A and support point B. In some cases there is more than one appropriate foundation. For specifications on foundation types a, b, and c refer to Figures 36, 37, and 38 and Tables XX, XXI, and XXII. Figures 36 through 38 illustrate the different foundation types.

(4) Determine connection details for members AC (AC') and BC to the foundation. Figure 39 is a side view of the foundation connection for members AC (AC') and BC. Figures 40 through 47 are top views of the foundation connections and base plates for various member shapes. Details for the connections may be developed from these figures and Tables VI and VII (for structural steel) and Tables XVIII and XIX (for cold-formed steel). See Section 4.7.1.3. for information on aluminum connections.

K. <u>If more than one row of structures</u> are used for the application, <u>proceed to Section 3.5.1 and follow</u> the method given to determine row spacing to avoid shading.

L. <u>If a portable (as opposed to dedicated) array structures</u> is to be <u>utilized for the application, proceed as follows:</u>

(a) Determine anchoring requirements for portable structure due to wind.

(1) For one-tier structure, uplift and lateral loads at support points A and B due to wind are:

$$B'_x = R'/\tan\beta$$

$$B'_y = R'$$

$$A'_x = 2R' \sin\theta - R'/\tan\beta$$

$$A'_y = 2R' \cos\theta - R'$$

where all quantities with a prime refer to the portable structure and where

θ	tilt angle
β	internal angle at support point B between member BC and the horizontal reference passing through support points A and B

B_x', A_x' lateral loads at support points A and B due to wind

B_y', A_y' uplift loads at support points A and B due to wind

R' maximum reaction for the top and bottom beams due to wind

(2) For two-tier structure, uplift and lateral loads at support points A and B due to wind are:

$$B_x' = 2R'/\tan \beta$$

$$B_y' = 2R'$$

$$A_x' = 4R' \sin \theta - 2R'/\tan \beta$$

$$A_y' = 4R' \cos \theta - 2R'$$

(3) Determine W': uniform windload applied to top and bottom beams.

$$W' = \text{wind load} \times L'/2$$

where

L' height of one module

(4) Determine R': the maximum reaction for the top and bottom beams due to wind.

Single span beams

$$R' = W'L/2$$

where

L span length between support frames

Two continuous span beams

$$R' = W' \times L \times 1.25$$

Three continuous span beams

$$R' = W' \times L \times 1.10$$

Four continuous span beams

$$R' = W' \times L \times 1.143$$

(5) Determine uplift and lateral loads at B due to wind using the equations in step L.(a)(1) or L.(a)(2), above as appropriate.

(6) Determine uplift and lateral loads at A due to wind using the equations in step L.(a)(1) or L.(a)(2), above as appropriate.

(7) The weight or other anchoring mechanism required at support point B is that necessary to resist both uplift (B_y') and sliding (B_x') due to wind.

(8) The weight or other anchoring mechanism required to support point A is that necessary to resist both uplift (A_y') and sliding (A_x') due to wind.

(9) Anchoring of portable structures:

- One method for anchoring portable structures is to connect support points A and B together with spreader bars and then apply weights, such as sand bags, to the spreader bars (Figure 13D) to provide resistance to overturning and/or sliding due to wind. In this case, the weights should be concentrated as nearly as possible to the support points A and B. To determine the sections to be used for the spreader bars, calculate the maximum moment for the spreader bars as follows:

$$M_m = d \times u_m$$

where

M_m	maximum moment for spreader bar
d	distance between support point (A or B) and point where weight is applied to spreader bar (i.e., the moment arm)
u_m	maximum uplift load due to wind (i.e., the greater of B_y' and A_y')

Determine sections to be used for the spreader bars as follows; proceed to:

Table III for <u>structural steel</u>
Table IV for <u>aluminum</u>
Table V for <u>cold-formed steel</u>

Find the member in the table which has the smallest value of M_{max} which is larger than the M_m calculated for the spreader bar.

Connection of the spreader bars to members AC (or AC') and BC should be equivalent to frame connections at C.

- Another method for anchoring portable structures is to connect support points A and B to T (tee) pieces or plates and then anchor them to the ground or other structures with weights, pins, stakes, screws, or bolts (Figure 13C). In this case, the weights, pins, stakes, screws, or bolts used for anchoring shall be sized to resist both uplift and sliding due to wind. Connection of the T (tee) pieces or plates to members AC (or AC') and BC should be equivalent to those for dedicated structures.

<u>Note</u>: If portable structures are to be left in one place for a year or more it is recommended that the weights or other anchoring mechanism be sized to provide 1-1/2 times that required to resist uplift and sliding. In addition, if the structure(s) are to be relocated to sites outside the locality for which they were designed, they should be reevaluated for the wind conditions for the new site.

4.7.1.2 Properties and Specifications for Structural Steel

Design, fabrication, and construction shall conform to the AISC Manual of Steel Construction, 8th Edition.

A. Materials

Steel shapes, plates, and rods shall be ASTM A36*.

Steel pipe shall be ASTM A501.

Welds and welding symbols shall conform to AWS Standards; the required electrode shall be AWS A5.1 or A5.5, E70XX; or AWS 5.20, E70T-X as required by AWS.

Bolts and nuts shall be ASTM A307 unless indicated otherwise.

B. Construction

All steel shall be shop painted as specified by the AISC. After erection, all exposed steel shall be touched up as required.

C. Structural Steel Beam Splicing Details

The planar beam tables assume that the two, three, and four-span beams are continuous over the supports, i.e., the flexural moments are transferred over the supports. The longer array structures will reach lengths that are impractical to obtain. It will therefore be necessary to provide moment and shear splices as shown in Figure 17.

Splices can be made anywhere, but practical and structural considerations would dictate the following considerations:

(a) Use as few splices as possible
(b) Plan for equal lengths in beam pieces to be spliced
(c) Try to avoid splices in areas of maximum flexural stresses, i.e., at mid span and over supports.

The moment splices in this handbook are designed to transfer the full moment and shear capacity of the structural sections.

4.7.1.3 Properties and Specifications for Aluminum

Design, fabrication, and construction shall conform to the Aluminum Association's Aluminum Construction Manual, (Section 1, Specifications for Aluminum Structures) and Aluminum Soldering Handbook.

A. Materials

Aluminum shapes, plates, and rods shall be alloy 6061-T6.

*ASTM A242 or A588 may be substituted where high corrosion resistance is required.

Bolts shall be alloy 2024-T4.

Nuts shall be alloy 6061-T6. ASTM B316.

Anchor bolts and nuts shall be ASTM 479 or ASTM A307 (must be painted after erection). ASTM B211.

Soldering shall be zinc-base of reaction type flux.

Base plates shall be ASTM A36 (must be painted), or stainless steel ASTM A666 B, or weathering high strength low alloy steel ASTM A242.

B. Construction

Finishing is not necessary; application engineer may wish to have aluminum anodized.

C. Structural Aluminum Beam Splicing Details

The notes and figures for steel beam splicing are applicable to aluminum. Aluminum is soldered where the steel is welded.

D. Structural Aluminum Frame Connection Details

(a) One-Tier Structure

(1) Top or Bottom Beam and Support Connection

The connection diagrams are the same as those for steel. The bolt connecting the beam to the clip angle is to be 1.27 cm (1/2 in.) diameter for $R \leq 29356.8$ N ($R \leq 6.6$ kips) or 1.58 cm (5/8 in.) diameter for $R > 29356.8$ N ($R > 6.6$ kips). One bolt is required at end support and two bolts are required at inner support. When top beam is a wide flange, use a 1.27 cm (1/2 in.) diameter bolt in place of 1.58 cm (5/8 in.) diameter bolt used in steel. A 1.27 cm (1/2 in.) bearing plate is to be used for this connection. Always solder when steel diagram specified weld. This beam connection is valid for one- or two-tier structures.

The support connection may be soldered or bolted. It may be bolted for the following cases only:

Bolt diameter		R_{max}	
cm	in.	N	kip
1.27	1/2	12009	2.7
1.58	5/8	17792	4
1.9	3/4	26688	6

Note: Edge distance (distance from center of bolt to edge of material) = 2 x diameter of bolt. Distance between the center of two bolts = 2.5 x diameter of bolts.

Figure 31 illustrates the support connection when member BC is a pipe.

(2) Bracing and Bracing Connections
Bracing and bracing connections are the same as for steel, except use 0.5 cm (3/16 in.) aluminum plate in place of the 0.4 cm (1/8 in.) steel plate for the Type 3 connection illustrated in Figure 25.

(3) Foundation Connections at A and B
These connections are the same as those for steel, except substitute aluminum in place of the steel plates. Specify solder in place of weld.

(b) Two-tier Structure

(1) Top or Bottom Beam
This connection is the same as that for one-tier structures.

(2) Middle Beam
When beam is a T (tee) section, connection is the same as for steel. Bolt table changes are:

Number of bolts	Inner span			
	Bolt diameter		R_{max}	
	cm	in.	N	kip
1	1.27	1/2	6672	1.5
1	1.58	5/8	11120	2.5
1	1.9	3/4	16457	3.7
2	1.58	5/8	22684	5.1
2	1.9	3/4	33360	7.5

Number of bolts	End of span			
	Bolt diameter		R_{max}	
	cm	in.	N	kip
1	1.27	1/2	14678	3.3
1	1.58	5/8	23129	5.2
1	1.9	3/4	33360	7.5

When beam is a wide flange, connection is the same as for steel, except use 0.95 cm (3/8 in.) bearing plate in place of 0.64 cm (1/4 in.) for steel, and use 1.27 cm (1/2 in.) diameter bolt in place of 1/58 cm (5/8 in.) diameter bolt for steel. Specify solder in place of weld.

(3) Support Connection
When member AC' is a wide flange, connection is the same as that for steel; the only difference is member BC is never a wide flange, only angle or pipe. Specify solder in place of weld.

When member AC' is an angle or channel, connection is the same as that for steel. Optional bolt table changes are:

Number of bolts	Bolt diameter		R_{max}	
	cm	in.	N	kip
1	1.27	1/2	9340	2.1
1	1.58	5/8	14678	3.3
1	1.9	3/4	21795	4.9
2	1.58	5/8	29801	6.7
2	1.9	3/4	43590	9.8

Note: Edge distance (distance from center of bolt to edge of material) = 2 x diameter of bolt. Distance between the center of two bolts = 2.5 x diameter of bolt.

(4) Bracing and Bracing Connections
Bracing and bracing connections are the same as for steel, except use 0.5 cm (3/16 in.) aluminum plate in place of the 0.4 cm (1/8 in.) steel plate for the Type 3 connection illustrated in Figure 25.

(5) Foundation Connections at A and B
These connections are the same as those for steel, except substitute aluminum in place of the steel plates. Specify solder in place of weld.

4.7.1.4 Properties and Specifications for Cold-Formed Steel

Design, fabrication, and construction shall conform to the AISI Cold-Formed Steel Design Manual, 1977 Edition.

A. Materials

Steel shapes shall have a minimum yield point of 33 ksi. ASTM A245 C, ASTM A446 A, ASTM A570 C, and ASTM A611 C may be considered.

Welds shall be A233 E60; weld rods and electrodes shall conform to specifications as required by the AWS.

Bolts and nuts shall be ASTM A307.

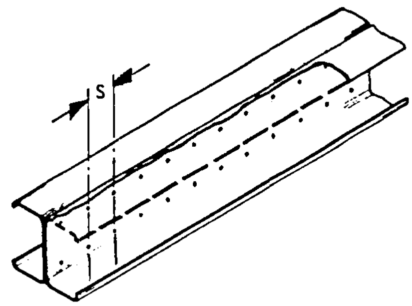
Angles, plates and rods shall be ASTM A36.

B. Construction

All steel shall be shop painted as specified by the AISC. After erection all exposed steel shall be touched up as required.

C. Connection of Two Channels Back-To-Back

There are to be two rows of 0.64 cm (1/4 in.) diameter bolts as shown in the sketch below. S_{max} is maximum allowable spacing of bolts. The distance between rows of bolts is determined by depth of channel. The following specifications apply to ASTM A307 bolts. Other possible connectors are rivets, spot welds, or self drilling/self tapping sheet metal screws; a structural engineer must be consulted for appropriate spacing of these alternate connectors.



(a) Beams and member AC

At supports S_{max} = 2.54 cm (1 in.)

Between supports S_{max} is the smaller of L/6 or 83.82 cm (33 in.)

where

L   span length of beam

or

L = 3L'/2 for member AC

and

L'   nominal distance between beams

(b) Member BC

Use Table XIII.

4.7.1.5 Foundation Design Specifications and Information Section

The following general notes are to be followed:

A. All design, fabrication and construction shall conform with the pertinent specifications of the American Concrete Institute's (ACI) Manual of Concrete Practice, 1977 Edition.

B. Steel reinforcing bars shall conform with ASTM 615 and be grade 60 ("Euronorm" grade 50).

C. Concrete strength shall be approximately 2070 N/cm^2 (3000 lb/in.^{2}) at 28 days and contain 6 percent air-entrainment. Mix proportions shall conform with ACI 211. Air-entrainment admixtures shall conform with ACI 212.

D. Concrete aggregates shall conform with ACI 221 (ASTM C33).

E. Cement shall be portland cement conforming with ASTM C-150, type I or type III.

F. Earth forms may be used where practical, except that foundation Type b shall have plastic film (ASTM C171) placed on both sides of the earth form and at the bottom prior to concrete placement to prevent migration of water from concrete to earth.

G. Wood or metal forms may be removed after one day and continuous moisture curing shall continue for not less than seven days.

H. Curing shall conform with pertinent ACI standards.

I. Photovoltaic array and structure shall not be erected until after seventh day of concrete placement.

J. Foundations depicted in this handbook are to be used in lieu of a soil investigation and foundation recommendation, and, consequently are likely to be in some cases overly conservative. Nevertheless, for earth of organic clays and peat (soil classifications PL, OH, PT) the foundations given in this handbook may be inadequate, and a soil investigation should be obtained.

K. The soil design criteria used for the foundations depicted in this handbook for clay, sandy clay, silty clay and clayey silt (CL, ML, MH and CH) with presumptive allowable bearing pressure = 47,880 N/m^2 (1000 psf), presumptive allowable lateral bearing pressure = 15,710 N/m^2 per meter of depth (100 psf per foot of depth) of natural grade, and a presumptive lateral sliding resistance = 6,225 N/m^2 (130 psf).

L. Massive crystalline bedrock, sedimentary and foliated rock, sand/gravel, gravel, sand, silty sand, clayey sand, silty gravel, and clayey gravel (SW, SP, SM, SC, GM, GC) all constitute soils of better quality than those for which the handbook foundations were designed. Therefore, the applications engineer may wish to have site-specific foundations designed by a professional civil/structural or soils engineer when the structure is to be sited on rock to save excavation and other costs. However, for soils other than massive bedrock the foundations depicted in this handbook, though conservative, would be little affected by better soil conditions since the foundations have been sized first for uplift and secondly proportioned for lateral loads and overturning, i.e., the foundation would be little affected in terms of volume of concrete required. In either case, should better soil conditions be present the best that could be hoped for would be elimination of the shear key and a more square-shaped foundation.

M. Backfill material may be on site material but must be soil or soil-rock mixture which is free from organic matter and other deleterious substance; it shall contain no rocks or lumps over 15 cm (6 in.) in greatest dimension and not more than 15 percent of the rocks or lumps shall be larger than 6.35 cm (2-1/2 in.) in greatest dimension. Backfill shall be placed in approximately 15-cm (6-in.) lifts and compacted. Compaction shall be at least 90 percent of standard proctor. (ASTM Designation D-698).

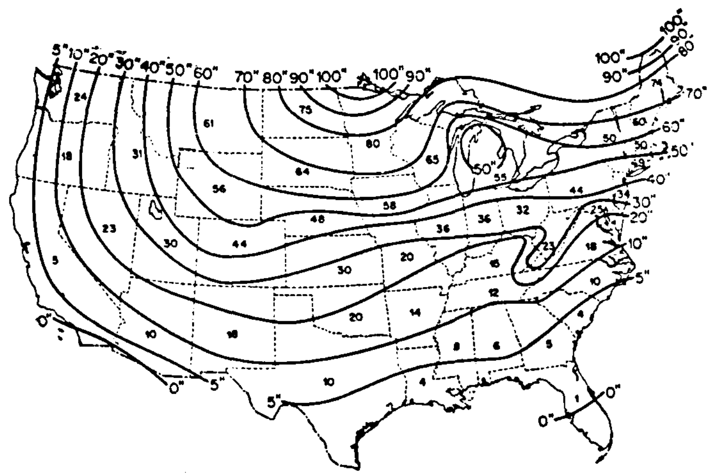


Figure 15. Extreme Frost Penetration, Inches Based on State Averages

N. Overexcavation: Should overexcavation occur, the overexcavation area shall be backfilled as indicated in item M above.

O. Cast-in-place auger piles may be earth formed provided there is no or little caving in of the earth walls. Should caving in occur, then forms shall be used.

P. Dewatering: Foundations shall not be placed when excessive water is present. Dewatering of foundation site will be required when water is present.

Q. In all cases the bottom of the foundation, not including the shear key, i.e., the foundation's vertical load bearing surface, shall be placed below local frost penetration. The designer should consult the local building code for the required minimum depth for foundations. In lieu of a local building code requirement, the designer should place the foundation of the structure below the extreme frost penetration indicated in Figure 15 for sites located within the continental U.S. For sites located outside the continental U.S. refer to Section 7.5 of this handbook for frost penetration at selected locations. For sites not covered in either Figure 15 or Section 7.5 (e.g., Southern Hemisphere sites), the designer will need to obtain frost penetration data from the local area or from other sources.

R. The foundation design tables of this handbook provide optional pedestal heights, i.e., heights above ground, of 15.2 cm to 76.2 cm (0.5 ft to 2.5 ft). This allows optional height selection of the foundations for snow, flood, row stepping to minimize shading in multiple row array layout designs, or for aesthetic considerations. There is also the option of designing the foundation for a particular pedestal height, then burying the foundation the height of the pedestal.

4.7.1.6 Design Figures for Triangular Framing Systems

This section consists of figures used to design triangular framing systems of the types covered by this handbook. Dimensions are in cm unless indicated otherwise.

Units: CM Unless Otherwise Noted

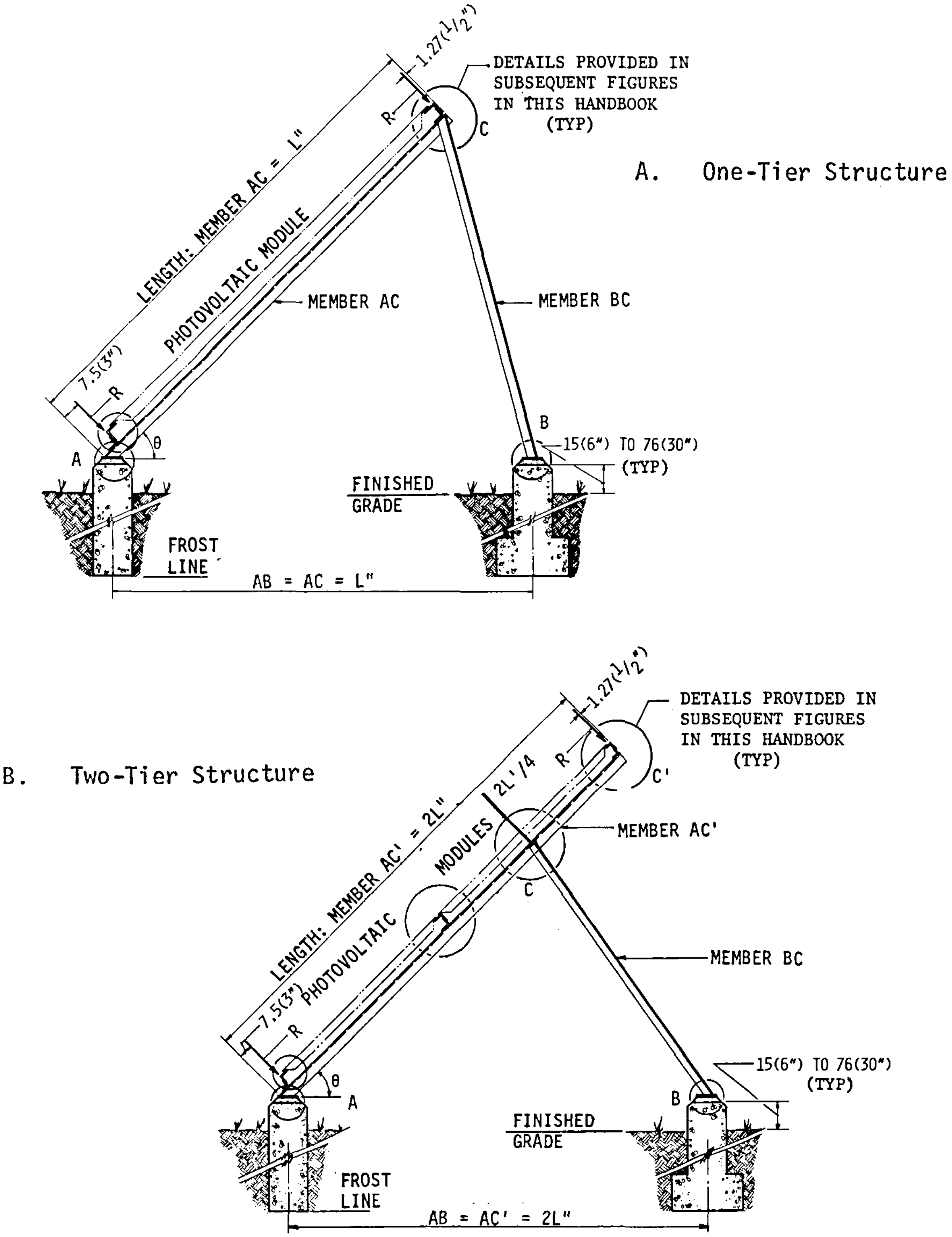
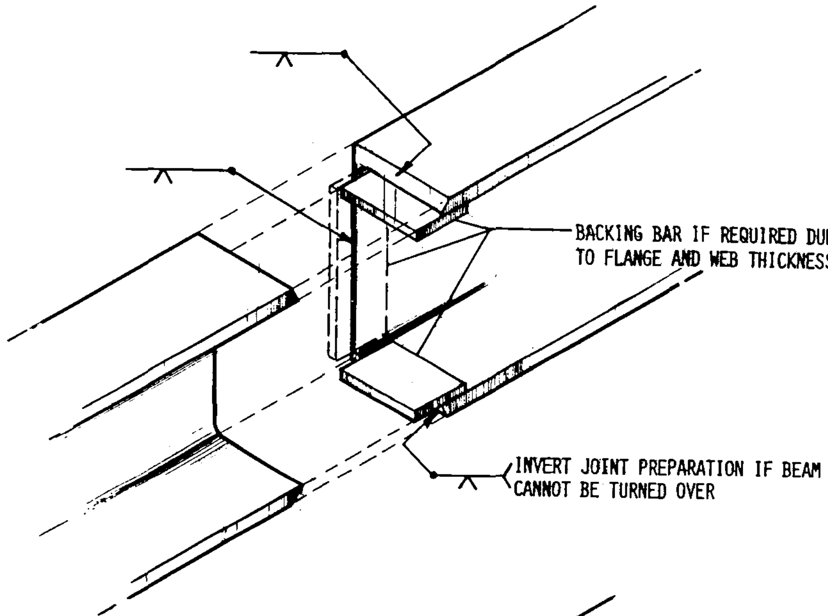
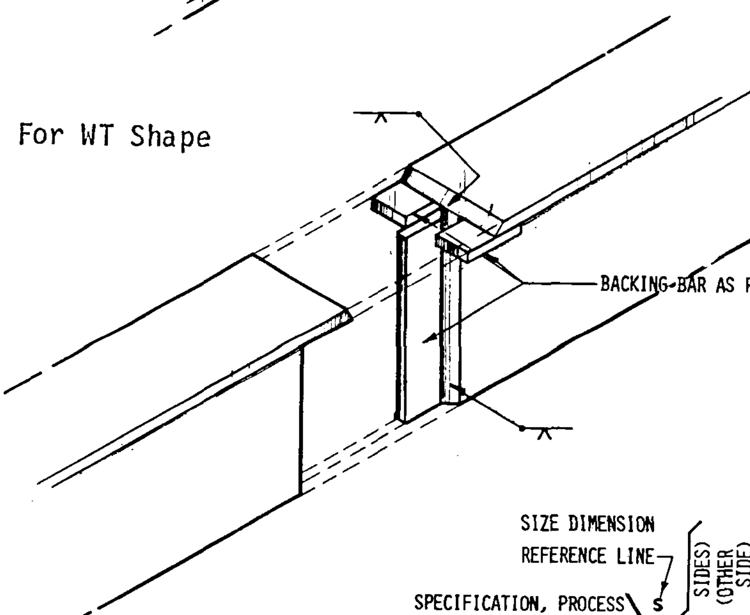


Figure 16. Dedicated Triangular Frame Structural Systems

A. For Channel, Angle or W Shape



B. For WT Shape



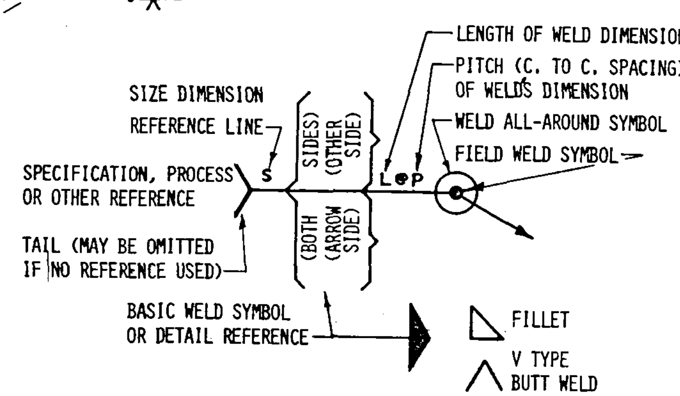
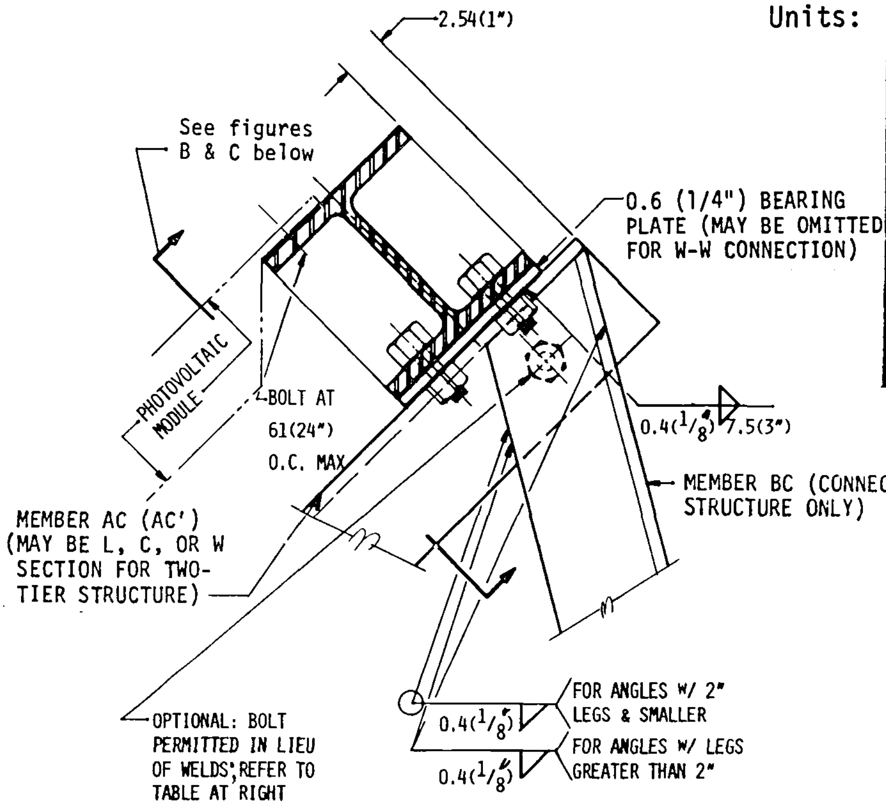


Figure 17. Structural Steel Beam Splicing Details

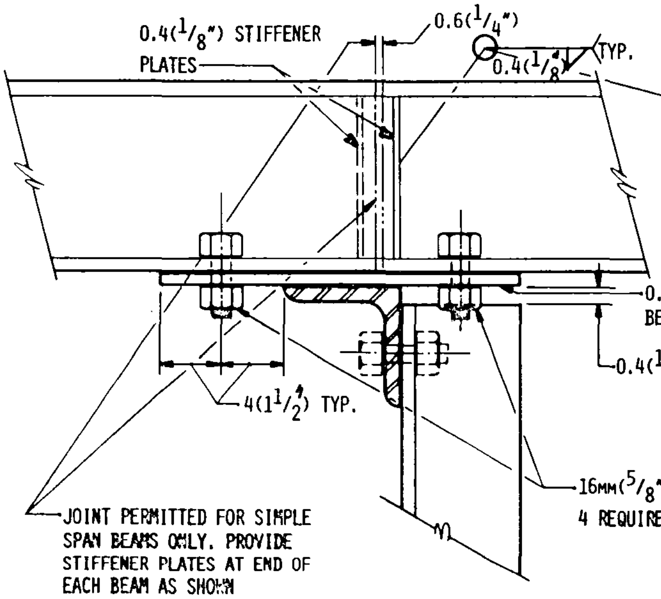
Units: CM Unless Otherwise Noted

BOLT TABLE					
REACTION ≤		ANGLE LEG		BOLT DIAM. (ϕ)	
N	K	CM	IN	MM	IN
8896	2	6.4	$2\frac{1}{2}$	10	$\frac{3}{8}$ (1)
8896	2	7.7	3	13	$\frac{1}{2}$ (2)
13344	3	7.7	3	16	$\frac{5}{8}$ (2)
17793	4	8.9	$3\frac{1}{2}$	16	$\frac{5}{8}$ (2)

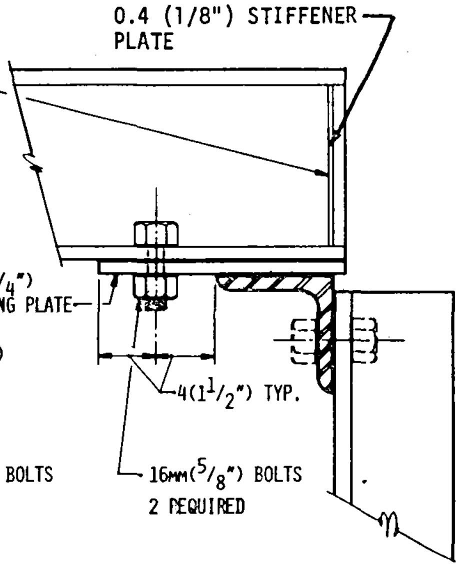
(1) Steel Type ASTM: A-325
(2) Steel Type ASTM: A-307



A. Beam Connection Detail at C, Side View.



B. Beam Connection Detail at C, Front View at Interior Support.



C. Beam Connection Detail at C, Front View at End Support.

Figure 18. One- and Two-Tier Structural Steel Frame Connection Details for W Section

Units: CM Unless Otherwise Noted

BOLT TABLE					
REACTION ≤		ANGLE LEG		BOLT DIAM. (ø)	
N	K	CM	IN	MM	IN
8896	2	6.4	$2\frac{1}{2}$	10	$\frac{3}{8}$ (1)
8896	2	7.7	3	13	$\frac{1}{2}$ (2)
13344	3	7.7	3	16	$\frac{5}{8}$ (2)
17793	4	8.9	$3\frac{1}{2}$	16	$\frac{5}{8}$ (2)

(1) Steel Type ASTM: A-325
(2) Steel Type ASTM: A-307

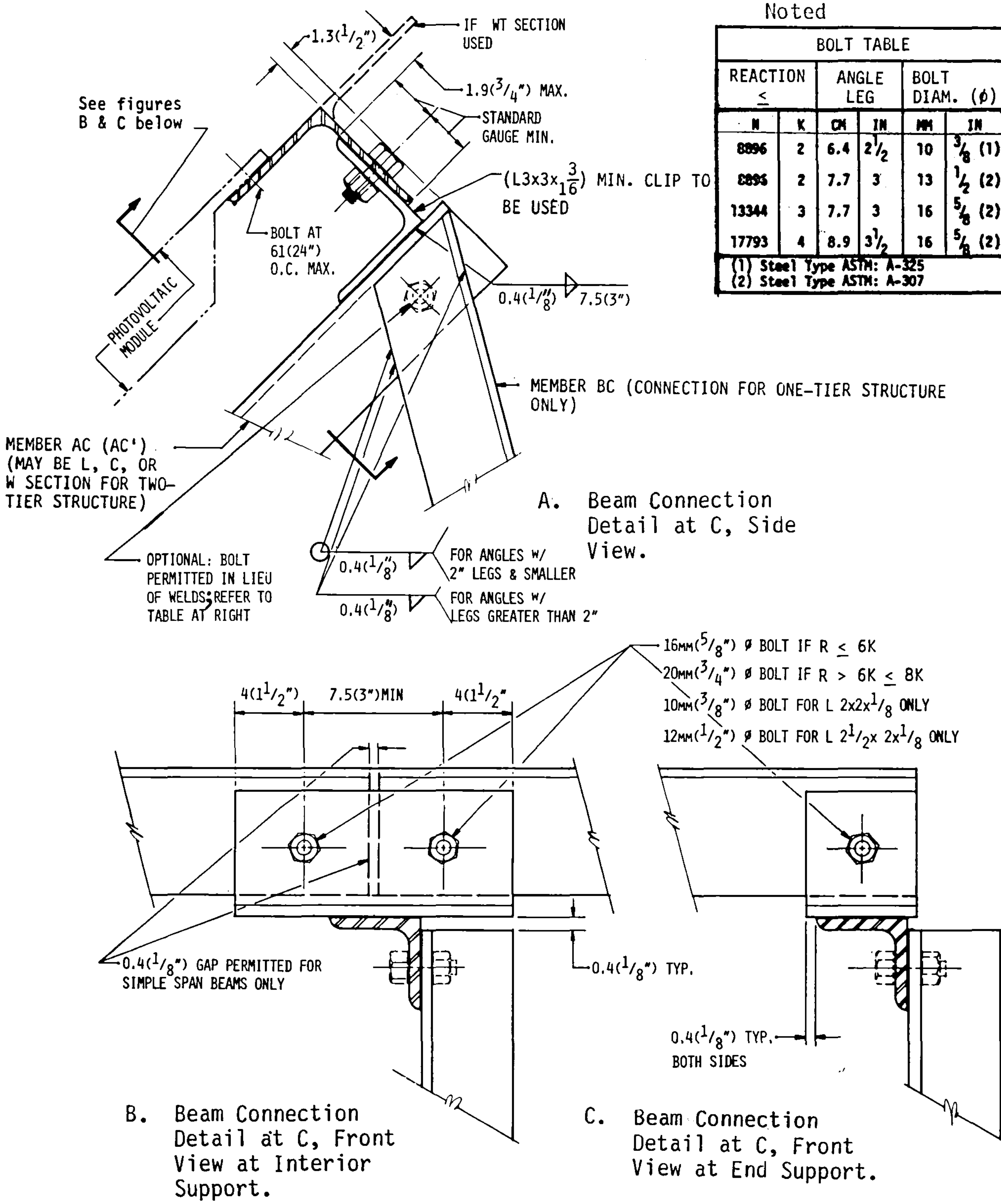
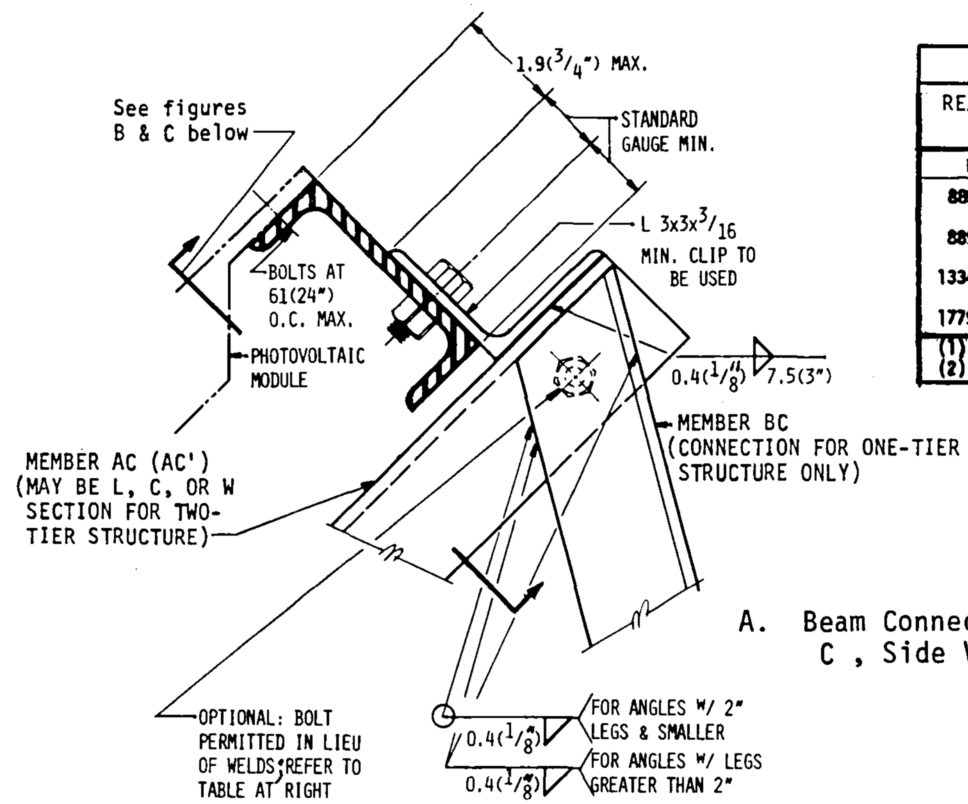


Figure 19. One- and Two-Tier Structural Steel Frame Connection Details for Angle or WT Section

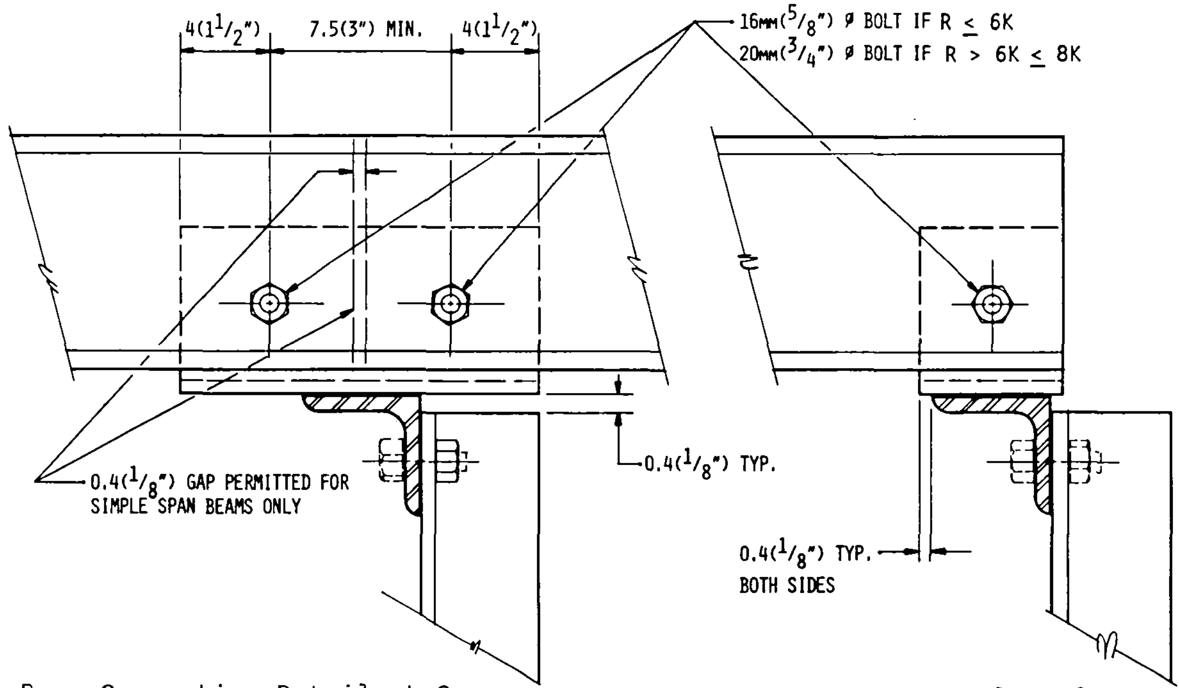
Units: CM Unless Otherwise Noted

BOLT TABLE					
REACTION ≤		ANGLE LEG		BOLT DIAM. (ϕ)	
N	K	CM	IN	MM	IN
8896	2	6.4	$2\frac{1}{2}$	10	$\frac{3}{8}$ (1)
8896	2	7.7	3	13	$\frac{1}{2}$ (2)
13344	3	7.7	3	16	$\frac{5}{8}$ (2)
17793	4	8.9	$3\frac{1}{2}$	16	$\frac{5}{8}$ (2)

(1) Steel Type ASTM: A-325
(2) Steel Type ASTM: A-307



A. Beam Connection Detail at C , Side View

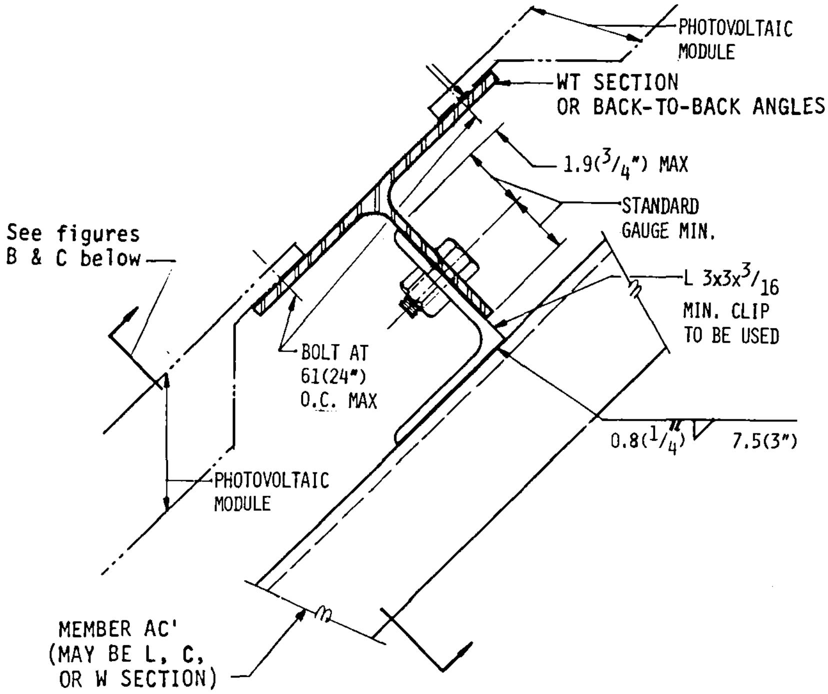


B. Beam Connection Detail at C, Front View at Interior Support

C. Beam Connection Detail at C, Front View at End Support

Figure 20. One-and Two-Tier Structural Steel Frame Connection Details for Channel

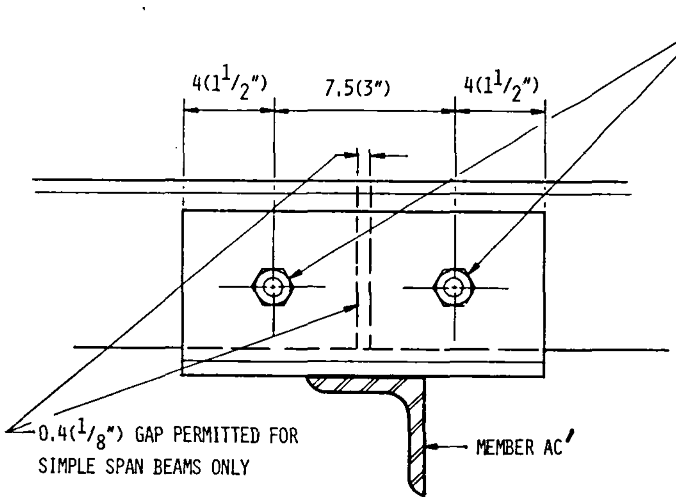
Units: CM Unless Otherwise Noted



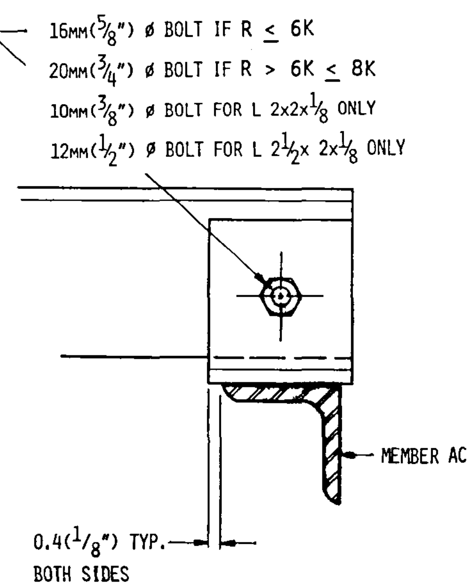
BOLT TABLE					
REACTION ≤		ANGLE LEG		BOLT DIAM. (ϕ)	
N	K	CM	IN	MM	IN
8896	2	6.4	$2\frac{1}{2}$	10	$\frac{3}{8}$ (1)
8896	2	7.7	3	13	$\frac{1}{2}$ (2)
13344	3	7.7	3	16	$\frac{5}{8}$ (2)
17793	4	8.9	$3\frac{1}{2}$	16	$\frac{5}{8}$ (2)

(1) Steel Type ASTM: A-325
(2) Steel Type ASTM: A-307

A. Beam Connection Detail at C, Side View.



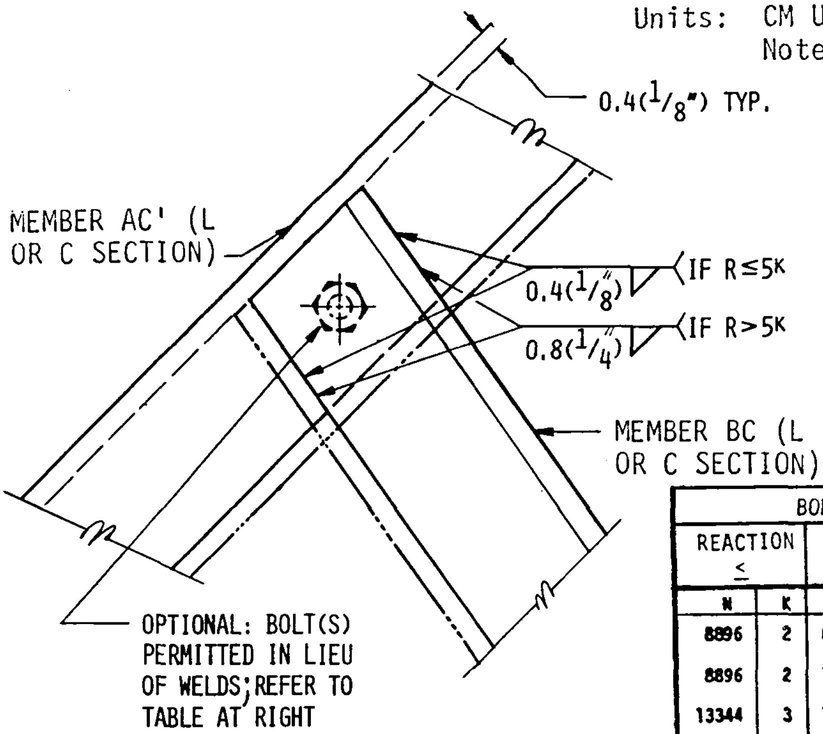
B. Beam Connection Detail at C, Front View at Interior Support.



C. Beam Connection Detail at C, Front View at End Support.

Figure 21. Structural Steel Frame Connection at Center Beam

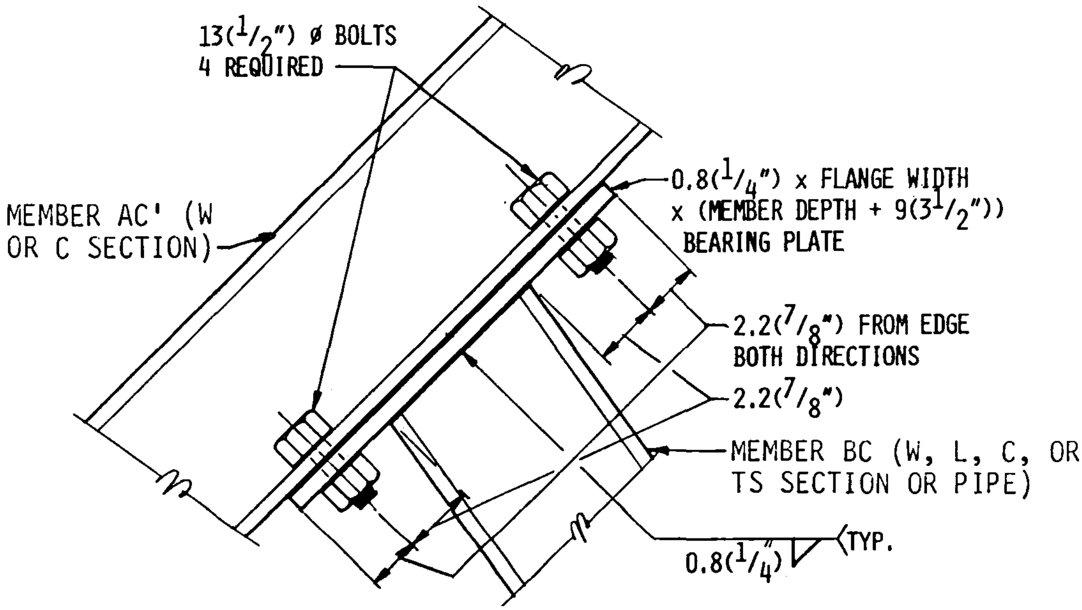
Units: CM Unless Otherwise Noted



BOLT TABLE					
REACTION ≤		ANGLE LEG		BOLT DIAM. (ϕ)	
N	K	CM	IN	MM	IN
8896	2	6.4	2 1/2	10	3/8 (1)
8896	2	7.7	3	13	1/2 (2)
13344	3	7.7	3	16	5/8 (2)
17793	4	8.9	3 1/2	16	5/8 (2)

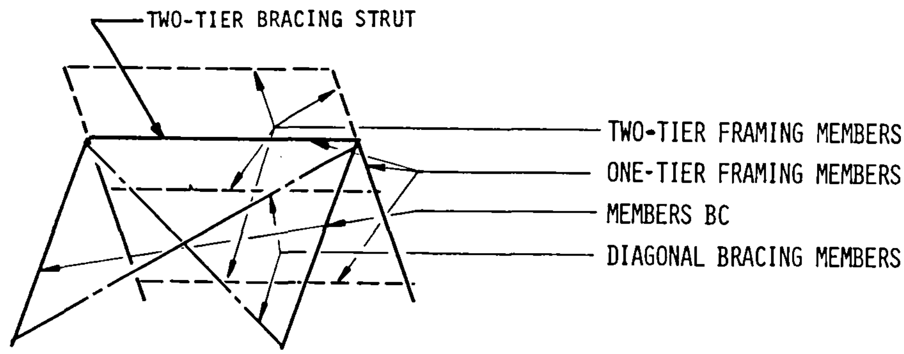
(1) Steel Type ASTM: A-325
(2) Steel Type ASTM: A-307

A. Connection at C When AC is Angle or Channel

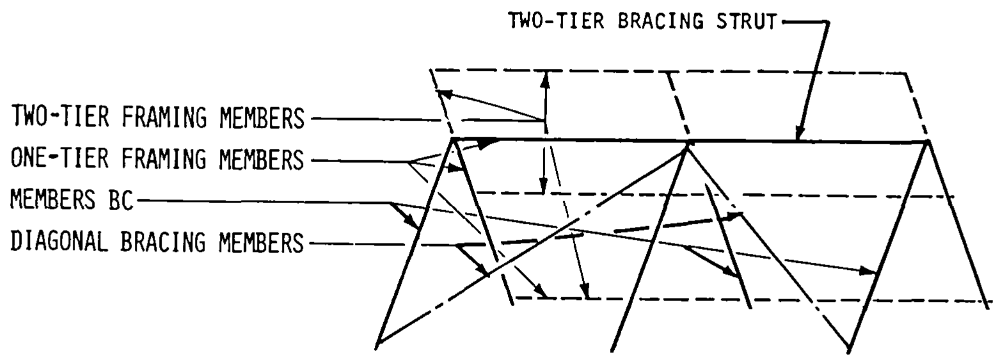


B. Connection at C When AC is W or C Section

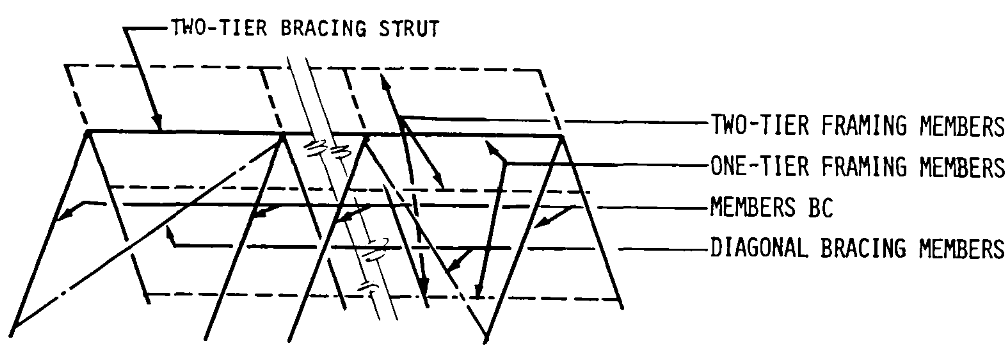
Figure 22. Two-Tier Structural Steel Connection of Member AC To Member BC



A. Bracing Diagram for 1 Bay, One- and Two-Tier Structures



B. Bracing Diagram for 2 Bay, One- and Two-Tier Structures



C. Bracing Diagram for 3 Bay and 4 Bay, One- and Two-Tier Structures

Figure 23. Bracing Diagrams

Units: CM Unless Otherwise
Noted

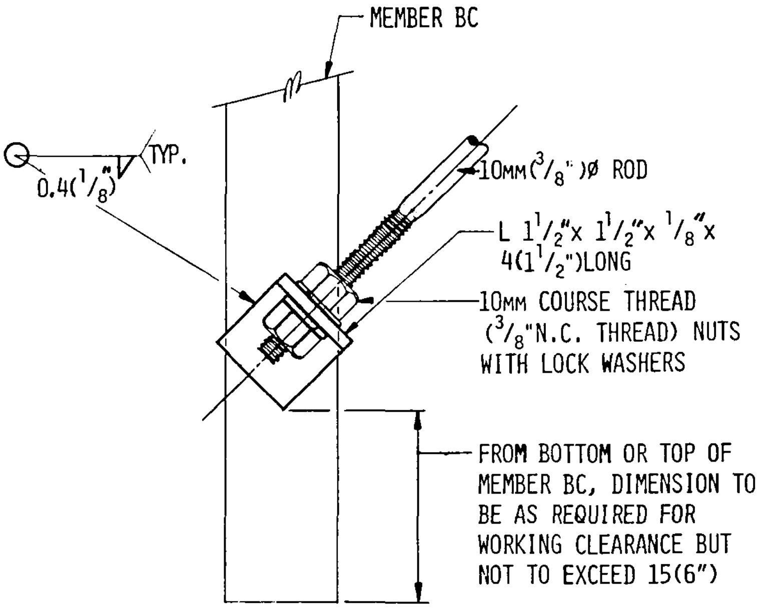


Figure 24A. Diagonal Bracing
Connection Detail Type 1

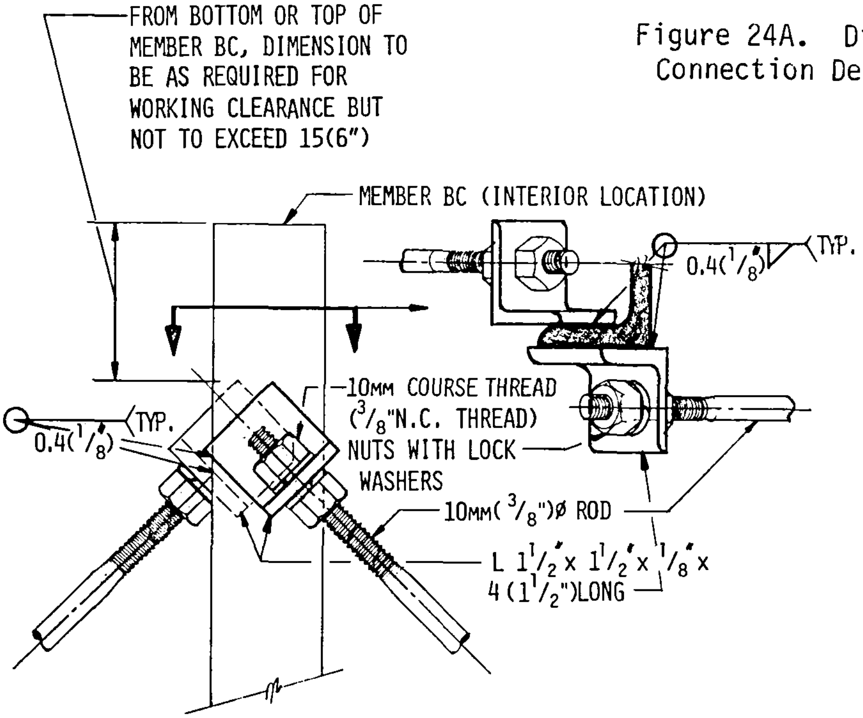


Figure 24B. Diagonal Bracing
Connection Detail Type 2

Units: CM Unless Otherwise Noted

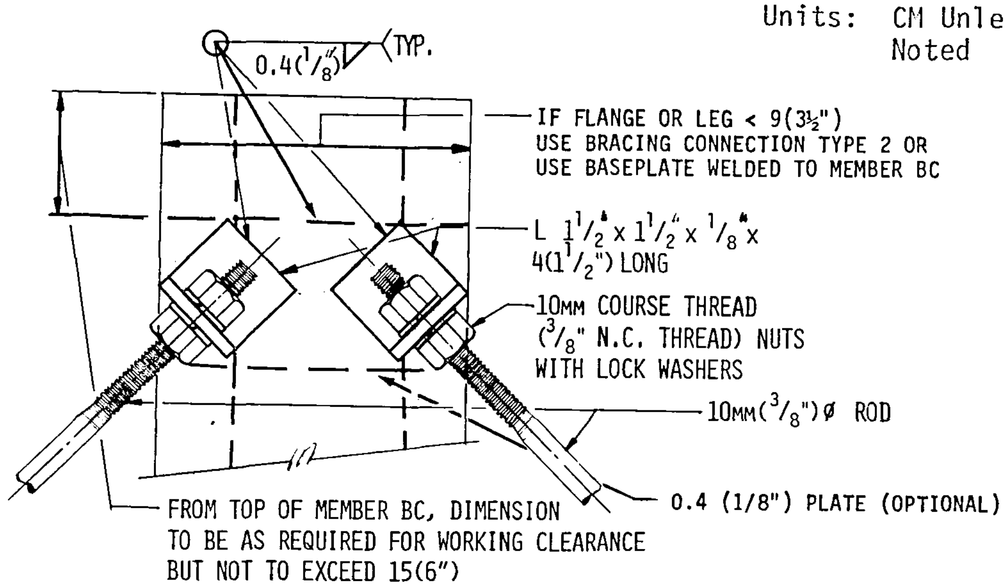


Figure 25. Diagonal Bracing Connection Detail Type 3

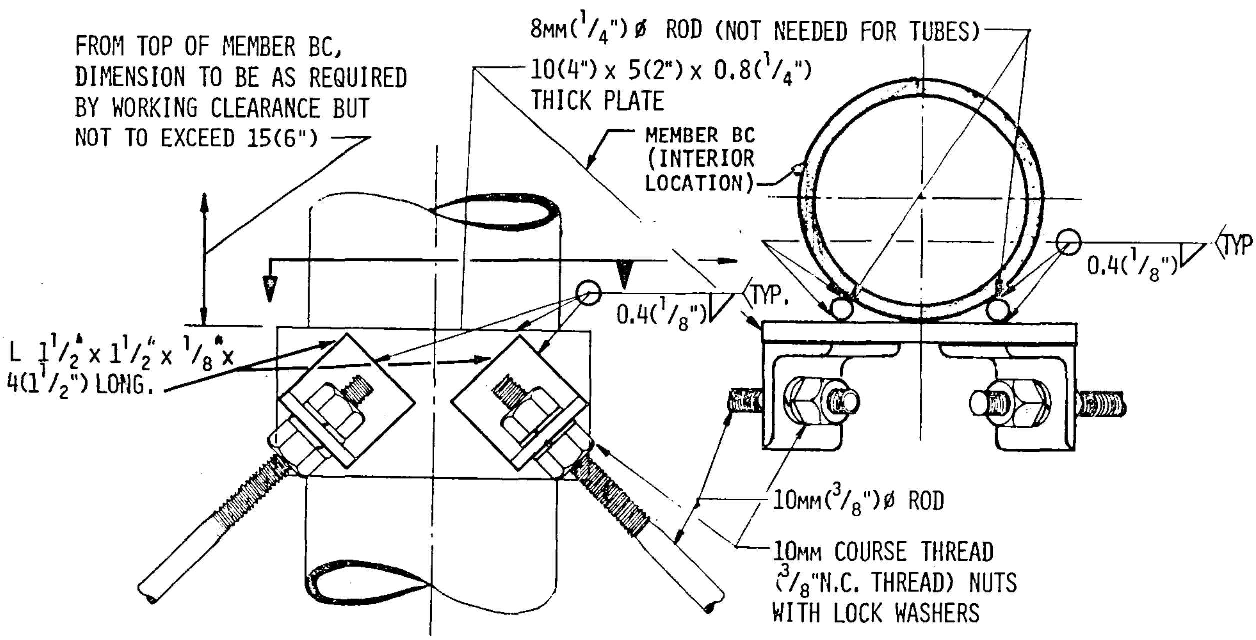


Figure 26. Diagonal Bracing Connection Detail Type 4 for Pipes and Tubes

Units: CM Unless Otherwise Noted

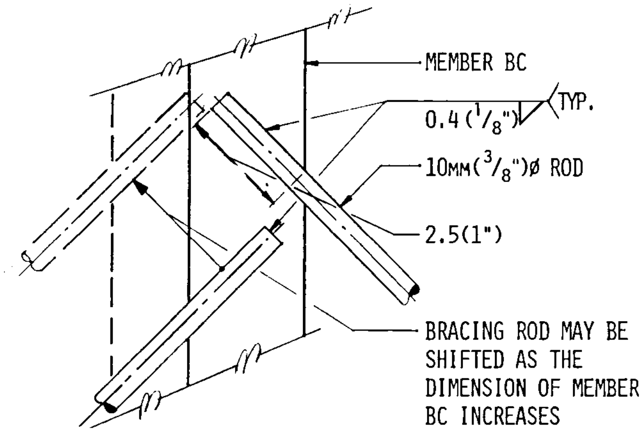


Figure 27. Diagonal Bracing Connection Detail Type 5

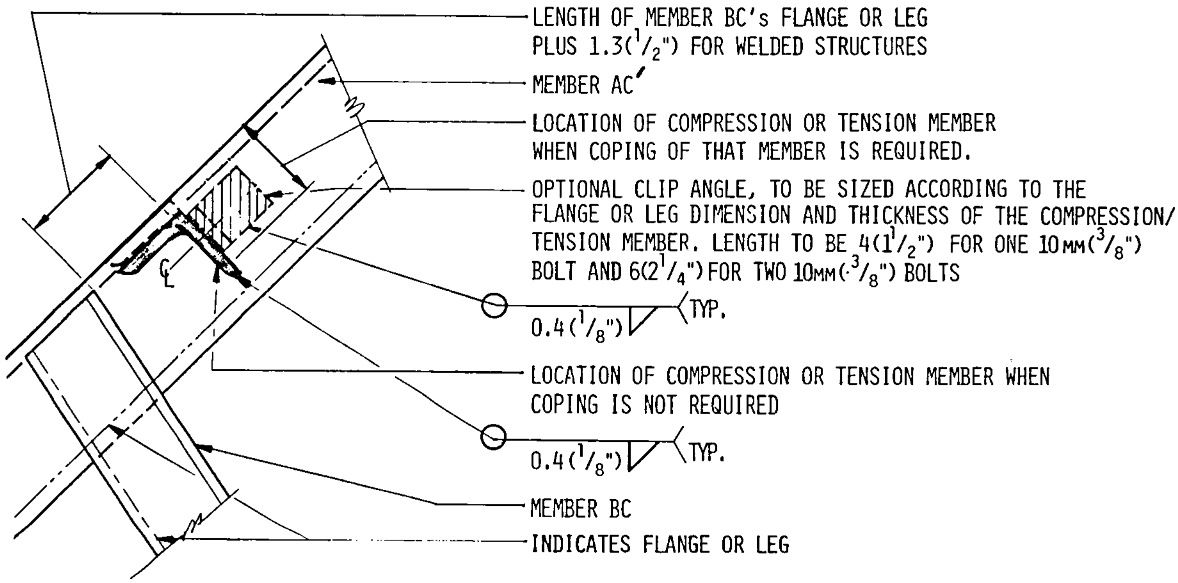


Figure 28A. Horizontal Bracing Strut Connection Detail Type 1 for Two-Tier Single Span Structures

Units: CM Unless Otherwise Noted

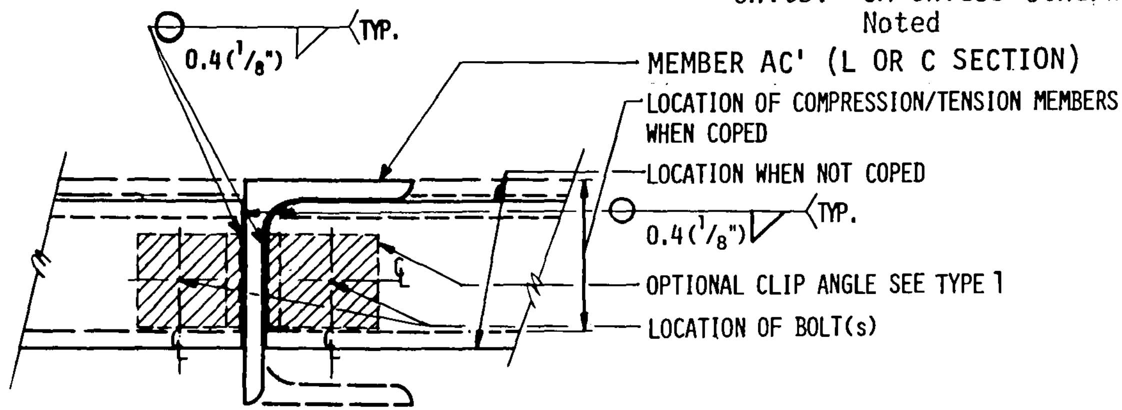


Figure 28B. Horizontal Bracing Strut Connection Detail Type 2 For Two-Tier Single Span Structures

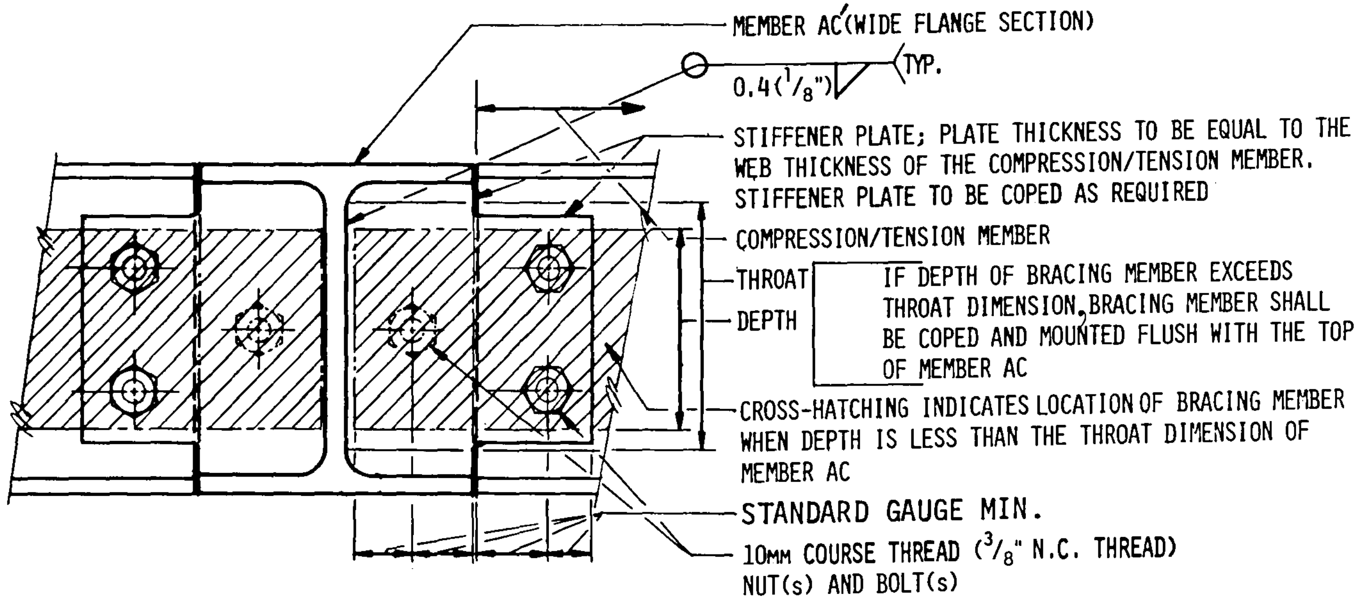


Figure 28C. Horizontal Bracing Strut Connection Detail Type 3 For Two-Tier Single Span Structures

Units: CM Unless Otherwise Noted

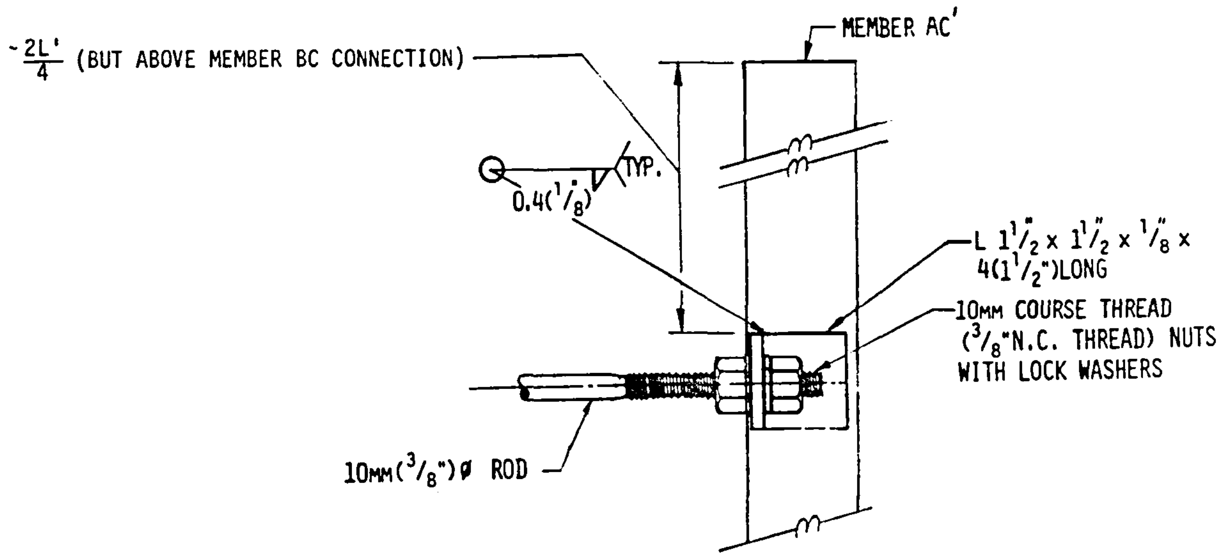


Figure 29. Horizontal Bracing Strut Connection Detail Type 1 for Two-Tier Two, Three, and Four Span Structures

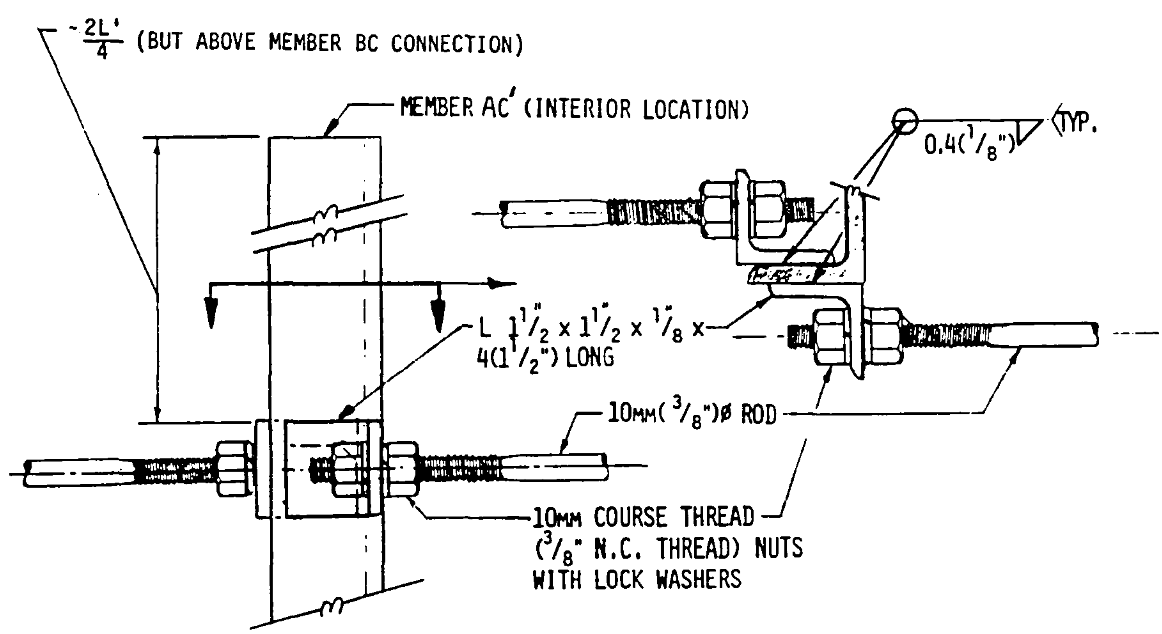
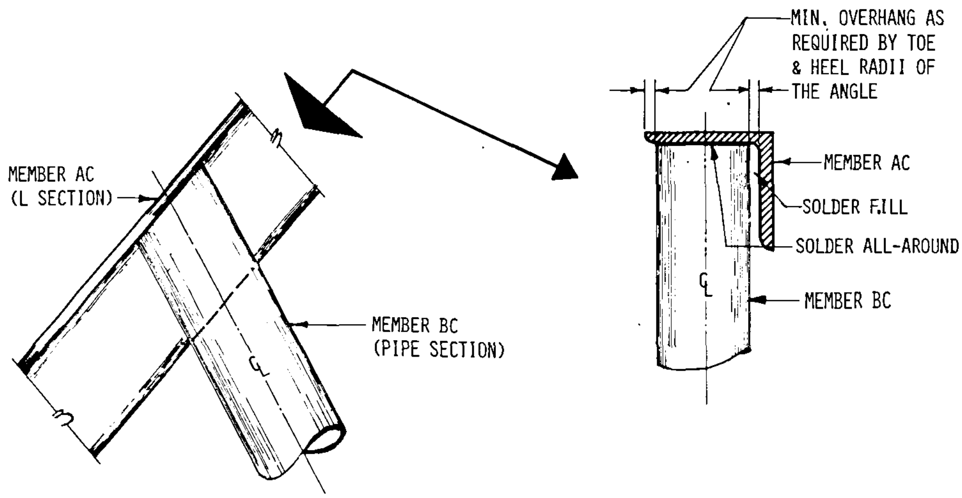
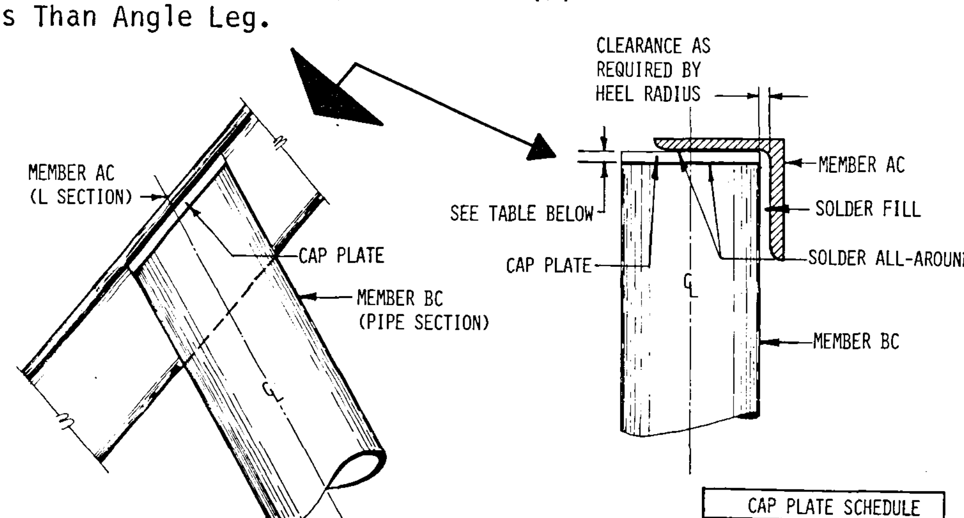


Figure 30. Horizontal Bracing Strut Connection Detail Type 2 for Two-Tier Two, Three, and Four Span Structures

Note: 1) When member AC' is a C, 2C, or W section, use Figure 25 with connection rotated to horizontal position.
2) When member AC' is a pipe or tube, use Figure 26 with connection rotated to horizontal position.



A. Connection at C Where Pipe Diameter (ϕ) Is Less Than Angle Leg.



B. Connection at C Where Pipe Diameter (ϕ) Is Greater Than or Equal to Angle Leg.

CAP PLATE SCHEDULE			
REACTION $\leq$		PLATE THICKNESS	
N	K	CM	IN
2224	0.5	0.635	1/4
4448	1	0.95	3/8
8896	2	1.27	1/2
17792	4	1.9	3/4
26688	6	2.2	7/8
35584	8	2.54	1

Figure 31. Structural Aluminum Frame Connection Details for Pipe to Angle Connection at C

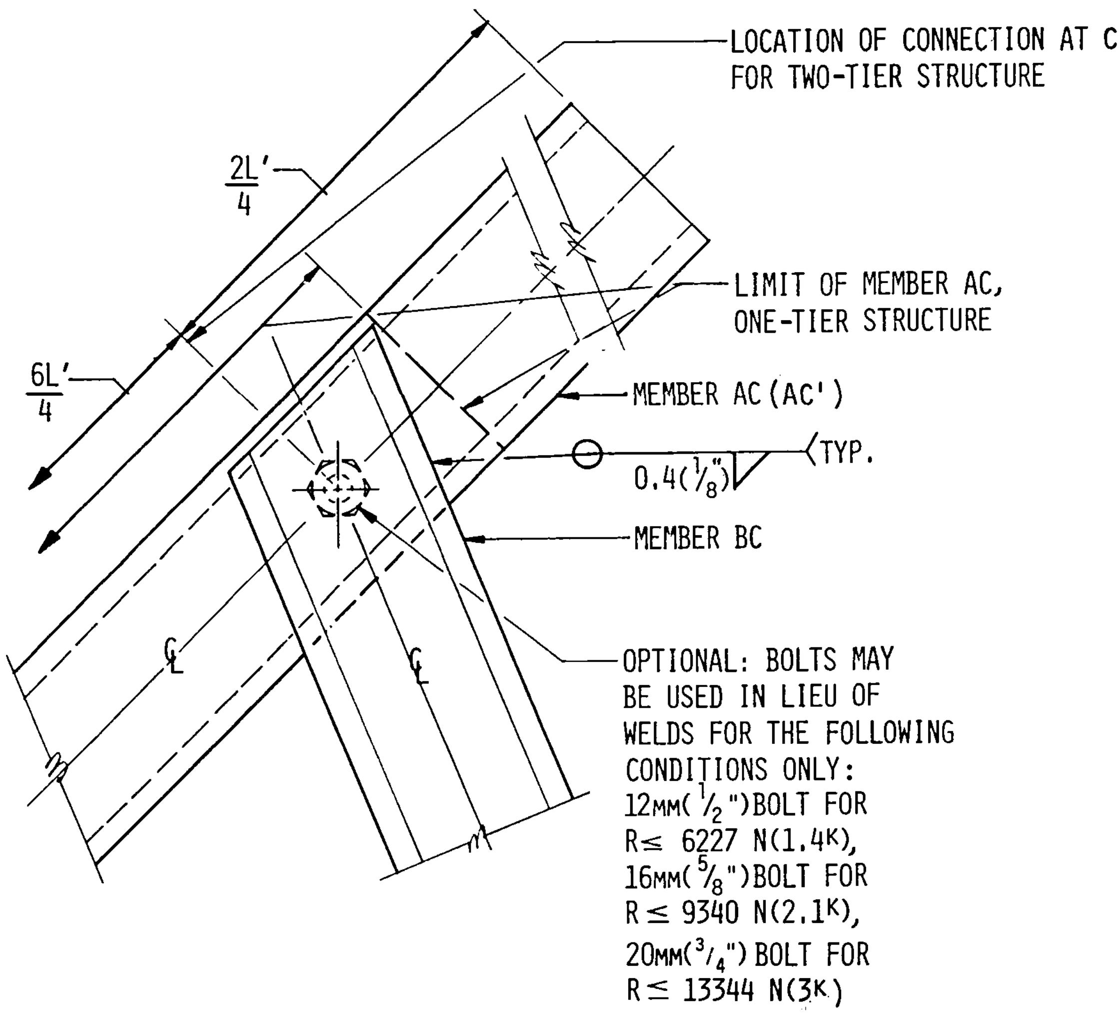
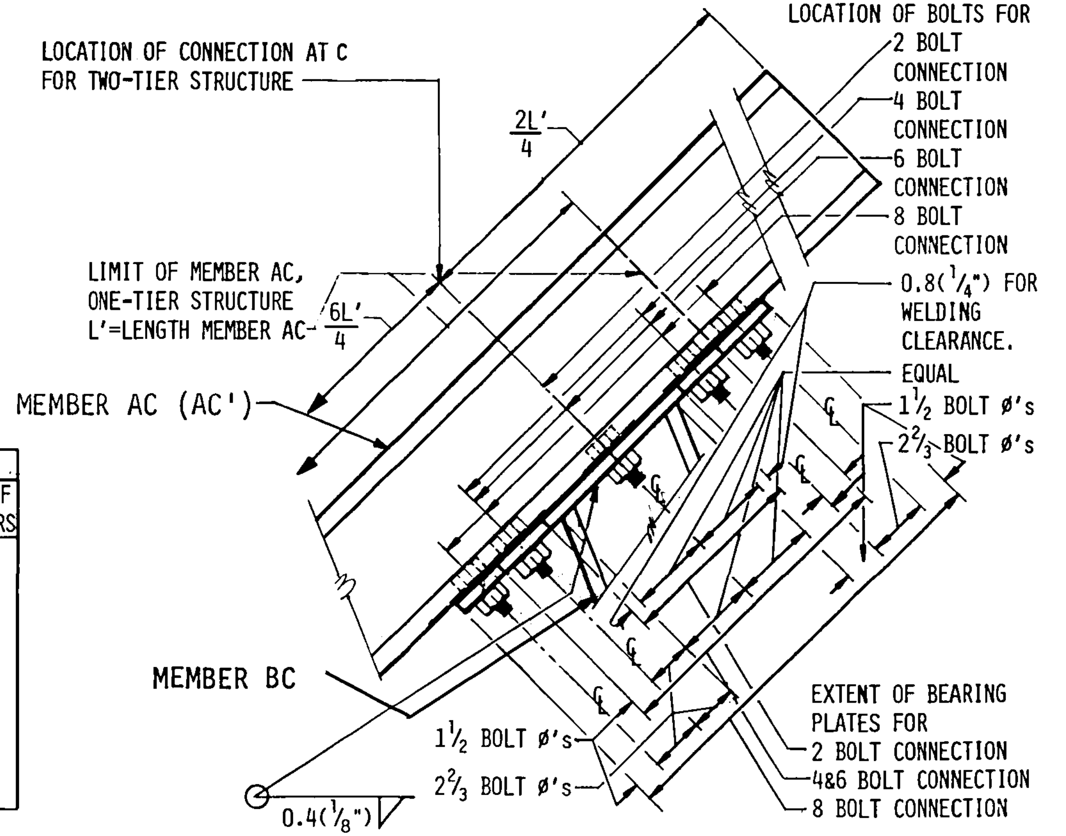


Figure 32. Cold-Formed Steel Support Connection at C for Single Channel Member AC and Single Channel Member BC

Units: CM Unless Otherwise Noted

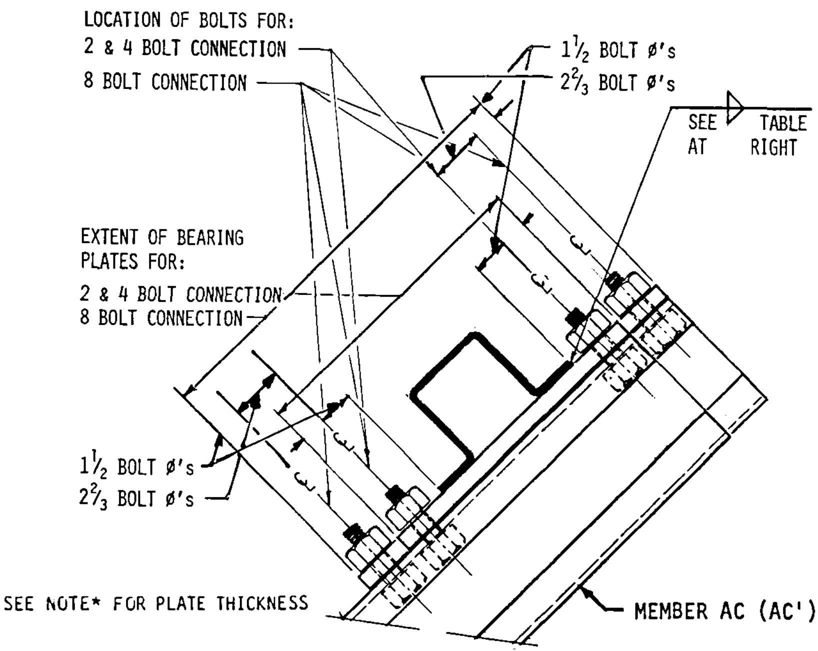
PLATE THICKNESS FOR ONE TIER STRUCTURE 0.8(1/4"), TWO-TIER STRUCTURE 13(1/4"); BEARING MAY BE ELIMINATED IF STRUCTURE DIMENSIONS PERMIT SHOP FABRICATION OR CERTIFIED WELDER IS AVAILABLE AT THE CONSTRUCTION SITE



BOLTING TABLE FOR SUPPORT CONNECTION ABOVE

REACTION ≤	BOLT Ø	# OF BOLTS	# OF BC CHANNELS	# OF TIERS
8006 N (1.8K)	8MM(1/4")	4	1	2
8896 N (2K)	8MM(1/4")	4	1	1
8896 N (2K)	10MM(3/8")	2	2	2
17792 N (4K)	10MM(3/8")	4	1	2
22240 N (5K)	10MM(3/8")	4	1	1
28912 N (6.5K)	10MM(3/8")	6	2	2
35584 N (8K)	10MM(3/8")	8	1	1
35584 N (8K)	10MM(3/8")	6	2	1
35584 N (8K)	10MM(3/8")	8	2	2

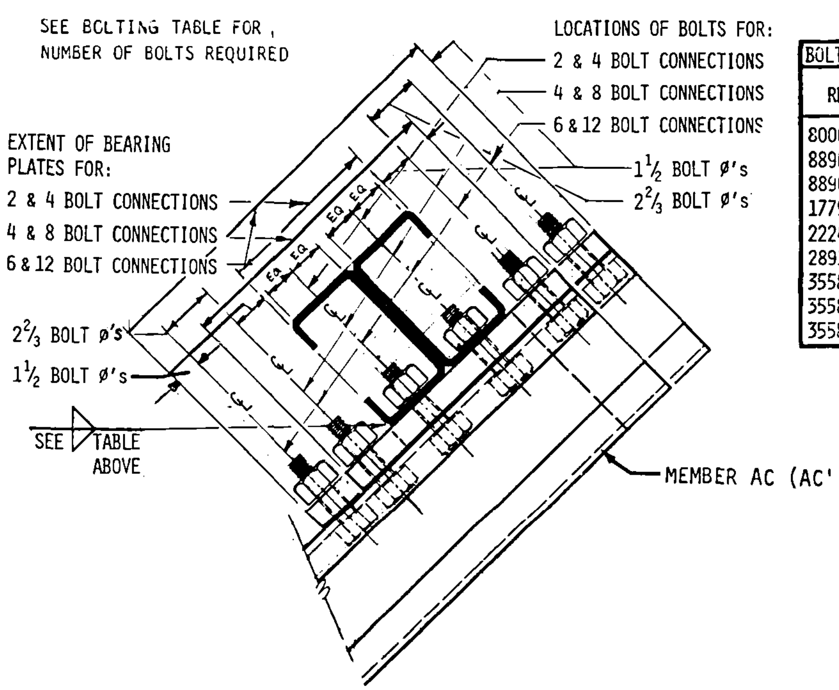
Figure 33. Cold-Formed Steel Support Connection at C for Double Channel Member AC and Single or Double Channel Member BC



REACTION ≤		WELD SIZE		WELD LENGTH	
(N)	(K)	(MM)	(IN)	(CM)	(IN)
6228	1.4	1.22	3/64	7.5	3
12899	2.9	1.22	3/64	15	6
8006	1.8	1.59	1/16	7.5	3
16013	3.6	1.59	1/16	15	6
10230	2.3	2.0	5/64	7.5	3
20461	4.6	2.0	5/64	15	6
14234	3.2	2.78	7/64	7.5	3
28467	6.4	2.78	7/64	15	6
18237	4.1	3.18	1/8	7.5	3
36474	8.2	3.18	1/8	15	6

*PLATE THICKNESS FOR: ONE-TIER STRUCTURE 0.8(1/4"), TWO-TIER STRUCTURE 13(1/4"), BEARING MAY BE ELIMINATED IF STRUCTURE DIMENSIONS PERMIT SHOP FABRICATION OR CERTIFIED WELDER IS AVAILABLE AT THE CONSTRUCTION SITE

Figure 34. Cold-Formed Steel Connection for HAT Section



BOLTING TABLE FOR SUPPORT CONNECTION				
REACTION ≤	BOLT ø	# OF BOLTS	# OF BC CHANNELS	# OF TIERS
8006 N (1.8K)	8MM(1/4")	4	1	2
8896 N (2K)	8MM(1/4")	4	1	1
8896 N (2K)	10MM(3/8")	2	2	2
17792 N (4K)	10MM(3/8")	4	1	2
22240 N (5K)	10MM(3/8")	4	1	1
28912 N (6.5K)	10MM(3/8")	6	2	2
35584 N (8K)	10MM(3/8")	8	1	1
35584 N (8K)	10MM(3/8")	6	2	1
35584 N (8K)	10MM(3/8")	8	2	2

Figure 35. Cold-Formed Steel Connection for Single or Double Channel Beam

Units: CM Unless Otherwise Noted

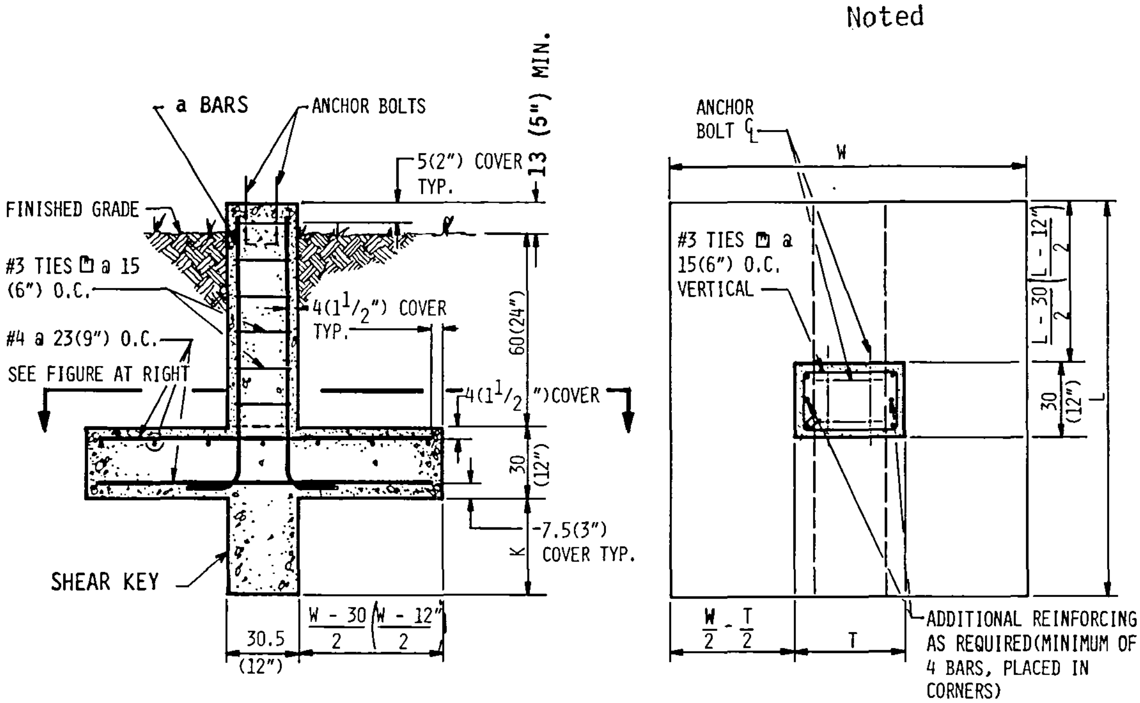


Figure 36. Foundation Type a (See Table XX)

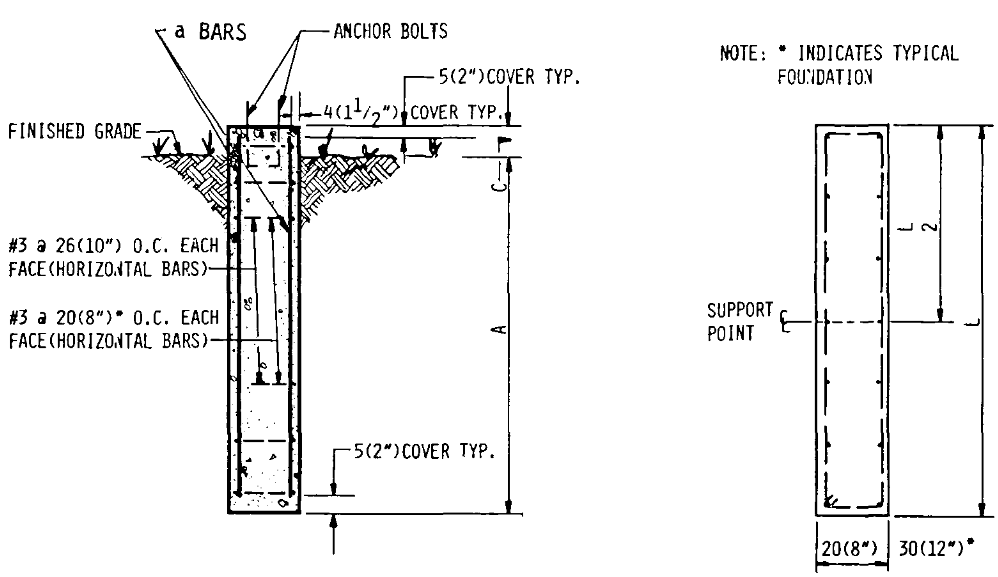


Figure 37. Foundation Type b (See Table XXI)

Units: CM Unless Otherwise Noted

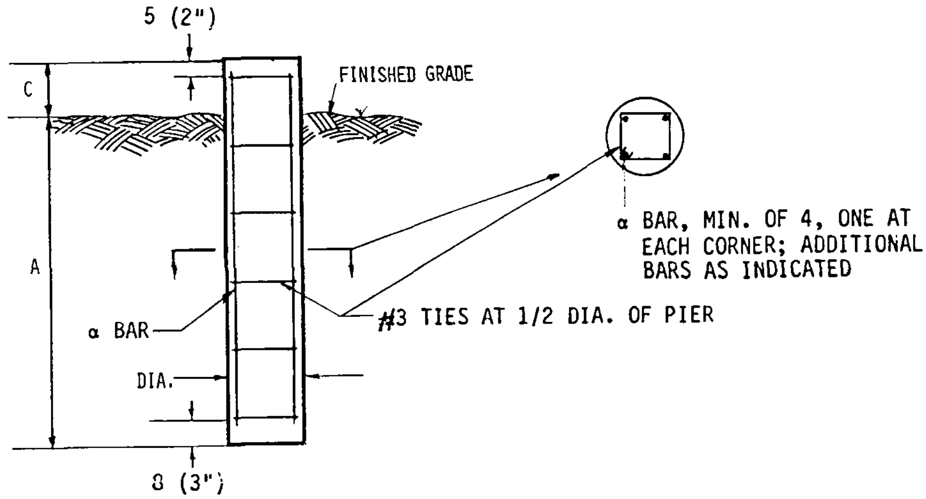


Figure 38. Foundation Type c (See Table XXII)

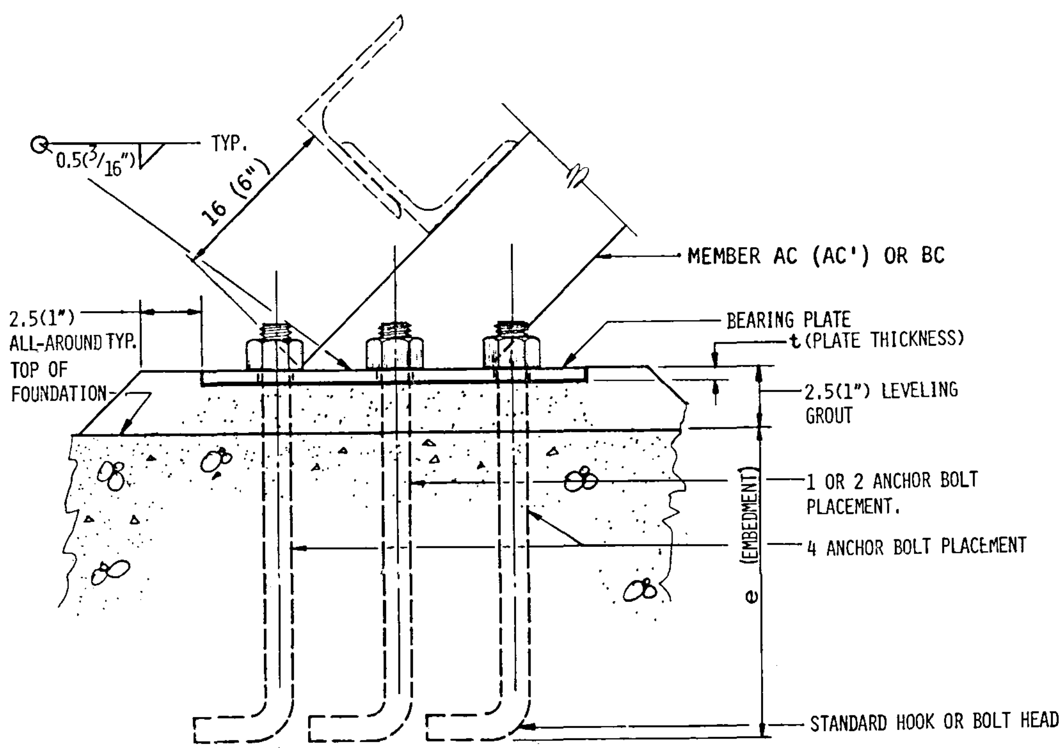


Figure 39. Foundation Connection Detail, Side View for Members AC and BC (See Tables VI and VII)

Units: CM Unless Otherwise Noted

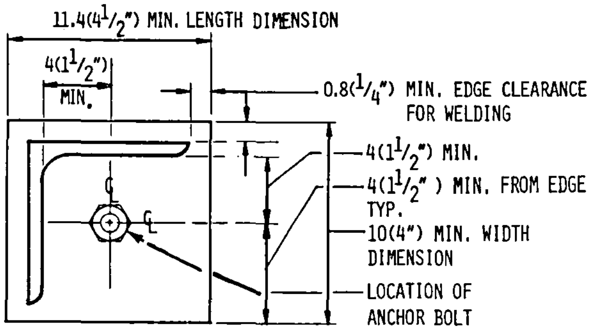


Figure 40. Foundation Connection Detail, Top View, Type 1

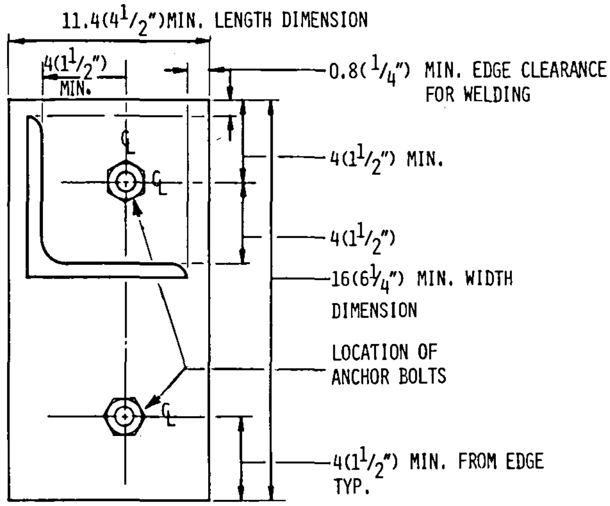


Figure 41. Foundation Connection Detail, Top View, Type 2

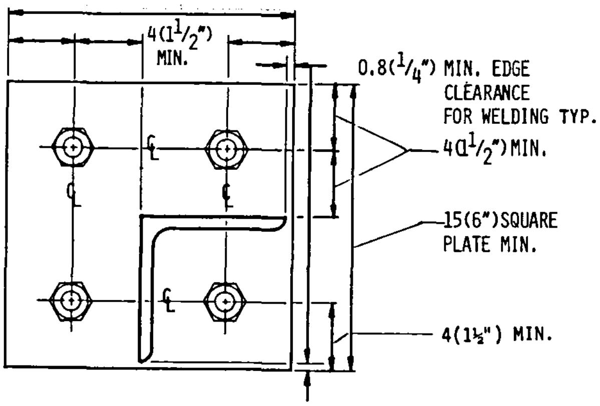


Figure 42. Foundation Connection Detail, Top View, Type 3

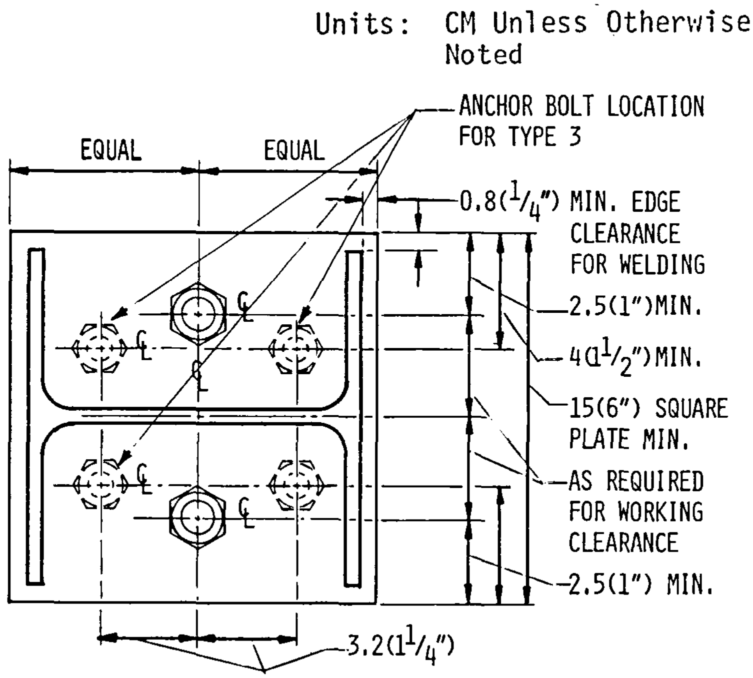


Figure 43. Foundation Connection Detail, Top View, Type 4

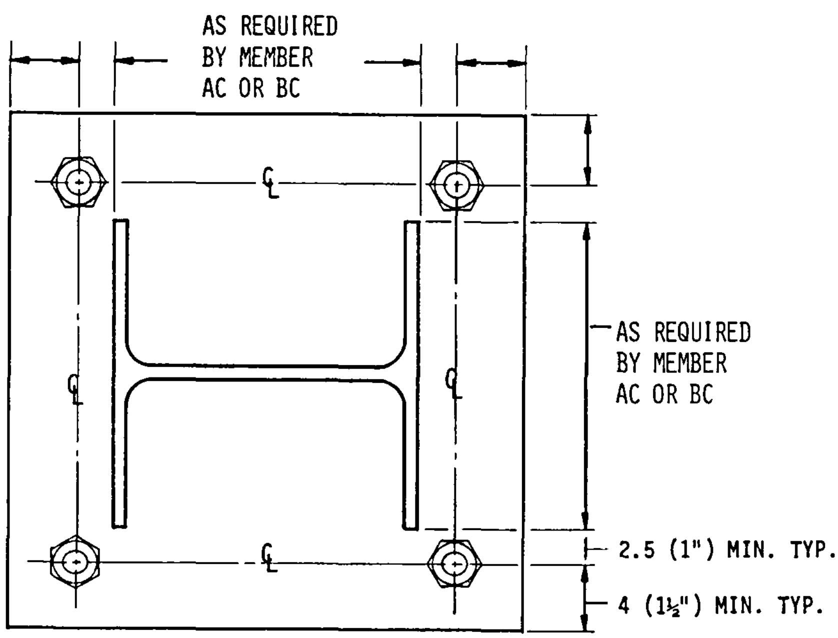


Figure 44. Foundation Connection Detail, Top View, Type 5

Units: CM Unless Otherwise Noted

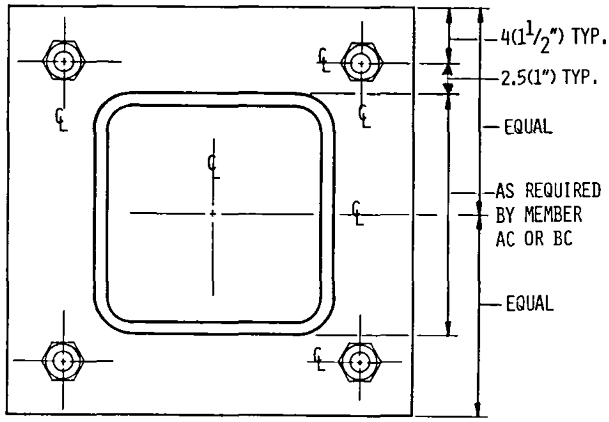


Figure 45. Foundation Connection Detail, Top View, Type 6

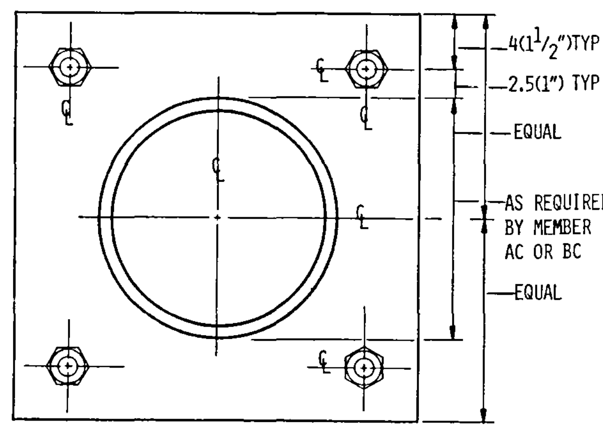


Figure 46. Foundation Connection Detail, Top View, Type 7

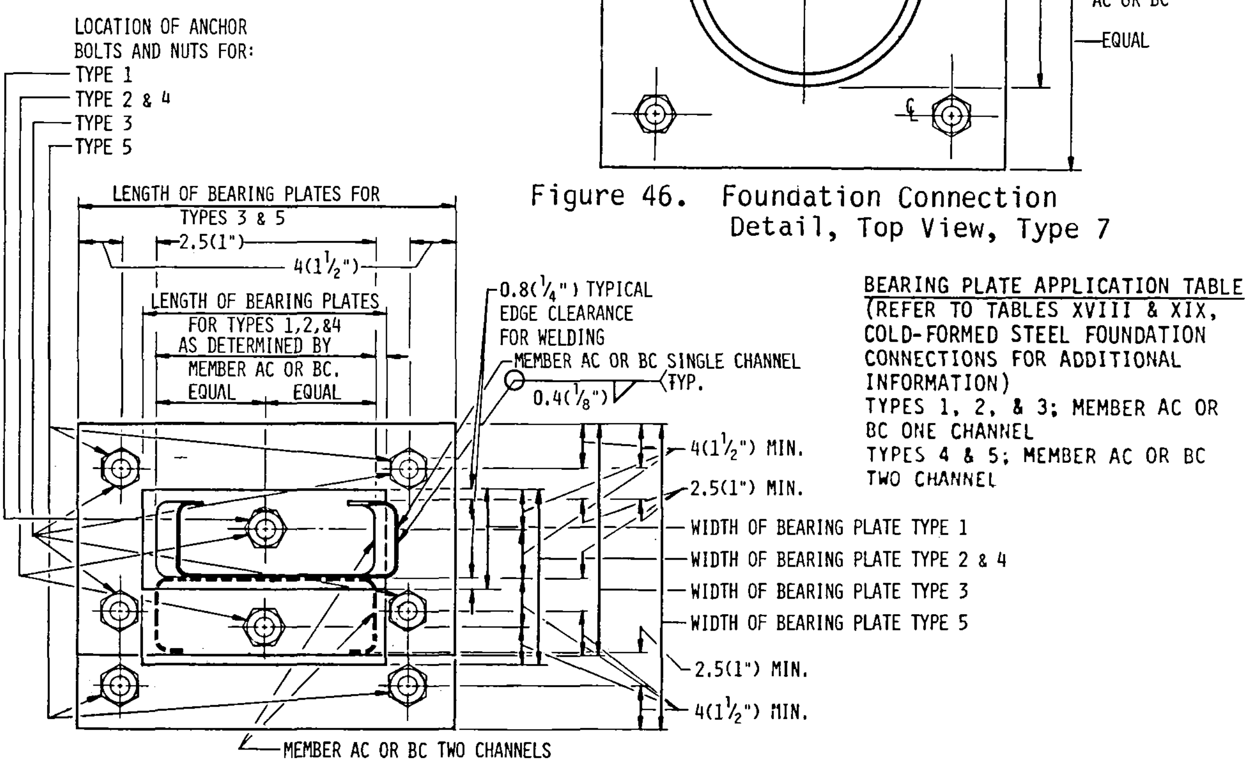


Figure 47. Foundation Connection Detail, Top View, Type 8

4.7.1.7 Design Tables for Triangular Framing Systems

This section consists of tables used to design triangular framing systems of the types covered by this handbook.

Shape (Section) Designations for the Structural Steel Tables are:

Shape	Designation	Explanation
I	W6x8.5	First number is nominal depth in inches. Second number is weight of section in lb/ft.
⊏	C4x5.4	Same as above.
T(tee)	WT4x5	Same as above.
○	Pipe 4 STD.	Number is nominal diameter of pipe in inches. STD means standard weight which designates the wall thickness of the pipe.
□	TS4x4x.375	TS stands for tube section, and first two numbers designate the nominal width and depth in inches. Last number is wall thickness in inches.
L	L2x2x$\frac{1}{4}$	First two numbers indicate length of legs in inches. Last number indicates thickness of legs in inches.

Shape Designations for the Aluminum Tables are:

Shape	Designation	Explanation
I	W6x4.16	First number is nominal depth in inches. Second number is weight in lb/ft.
⊏	C5x2.32	Same as above.
T (tee)	T2x1.26	Same as above.
L	L2-$\frac{1}{2}$x2-$\frac{1}{2}$x$\frac{1}{8}$	First two numbers indicate length of legs in inches. Last number indicates thickness of legs in inches.
○	Pipe 8#80	First number is nominal pipe diameter in inches. Second number is the schedule number which conforms to ANSI standard for wrought iron and wrought steel pipe, B36.10.

Note: Spaces occur in tables where Aluminum Sections are not adequate.

Shape Designations for the Cold-Formed Steel Tables are:

Shape	Designation	Explanation*
	C3x1.16	First number is nominal depth in inches. Second number is weight in lb/ft.
	2C3x2.31	Same as above
	H1.5x768	Same as above

Note: Spaces occur in tables where Cold-Formed Steel Sections are not adequate.

*Details to further specify shapes are included with the tables.

TABLE III. MAXIMUM MOMENT CAPACITY OF VARIOUS ASTM A36 STEEL SHAPES USED AS BEAMS

[M_{max} is maximum moment capacity; L_c is maximum unbraced length of compression member.]

UNEQUAL LEG ANGLES	WT/LENGTH		M_{max}		L_c	
	$\frac{N}{m}$	$\frac{lb}{ft}$	N-m	in.-k	m	ft
$L2\frac{1}{2}x2x\frac{3}{16}$	40.1	2.75	969	8.56	0.61	2
$L3x2x\frac{3}{16}$	44.7	3.07	1372	12.14	.61	2
$L3x2\frac{1}{2}x\frac{3}{16}$	49.4	3.39	1422	12.58	.61	2
$L2\frac{1}{2}x1\frac{1}{2}x\frac{5}{16}$	57.2	3.92	1467	12.98	.61	2
$L3x2x\frac{1}{4}$	59.8	4.1	1793	15.86	.61	2
$L3x2\frac{1}{2}x\frac{1}{4}$	65.6	4.5	1855	16.41	.61	2
$L3\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	71.4	4.9	2497	22.09	.61	2
$L3\frac{1}{2}x3x\frac{1}{4}$	78.7	5.4	2566	22.7	.61	2
$L4x3x\frac{1}{4}$	84.6	5.8	3307	29.26	.61	2
$L4x3\frac{1}{2}x\frac{1}{4}$	90.4	6.2	3407	30.13	.61	2
$L5x3x\frac{1}{4}$	96.4	6.6	5059	44.76	.61	2
$L5x3\frac{1}{2}x\frac{1}{4}$	102.2	7.0	5191	45.93	.61	2
$L6x3\frac{1}{2}x\frac{1}{4}$	115.3	7.9	7308	64.66	.61	2
$L6x4x\frac{1}{4}$	121.2	8.3	7473	66.12	1.22	4
$L6x3\frac{1}{2}x\frac{5}{16}$	143.1	9.8	9027	79.87	.61	2
$L6x4x\frac{5}{16}$	150.4	10.3	9226	81.63	1.22	4
$L6x3\frac{1}{2}x\frac{3}{8}$	170.8	11.7	10714	94.8	.61	2
$L6x4x\frac{3}{8}$	179.6	12.3	10979	97.14	1.22	4

EQUAL LEG ANGLES	WT/LENGTH		M_{max}		L_c	
	$\frac{N}{m}$	$\frac{lb}{ft}$	N-m	in.-k	m	ft
$L1x1x\frac{1}{8}$	11.7	0.80	102.7	0.91	0.61	2
$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	14.7	1.01	162.7	1.44	.61	2
$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	18.0	1.23	238.5	2.11	.61	2
$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	21.0	1.44	331.2	2.93	.61	2
$L2x2x\frac{1}{8}$	24.1	1.65	433	3.83	.61	2
$L1\frac{3}{4}x1\frac{3}{4}x\frac{3}{16}$	30.9	2.12	475	4.20	.61	2
$L2x2x\frac{3}{16}$	35.6	2.44	627	5.55	.61	2
$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	44.8	3.07	1003	8.85	.61	2
$L3x3x\frac{3}{16}$	54.1	3.71	1458	12.9	.61	2
$L3x3x\frac{1}{4}$	71.5	4.9	1907	16.87	.61	2
$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	84.6	5.8	2624	23.22	.61	2
$L4x4x\frac{1}{4}$	96.2	6.6	3472	30.70	1.22	4
$L4x4x\frac{5}{16}$	119	8.2	4261	37.70	1.22	4
$L4x4x\frac{3}{8}$	143	9.8	5026	44.47	1.22	4
$L5x5x\frac{5}{16}$	150	10.3	6746	59.69	1.22	4
$L5x5x\frac{3}{8}$	179	12.3	8002	70.80	1.22	4
$L6x6x\frac{5}{16}$	181	12.4	9822	86.90	1.83	6
$L6x6x\frac{3}{8}$	217	14.9	11673	103.28	1.83	6

WIDE FLANGE	M_{max}		L_c	
	N-m	in.-k	m	ft
W6x9	19730	177.50	0.61	2
W6x12	26156	231.42	.61	2
W6x16	36799	325.58	.61	2
W6x20	48344	427.72	.61	2
W6x25	60249	533.06	.61	2
W8x10	28132	248.9	.61	2
W8x13	35716	316.33	.61	2
W8x15	42572	376.65	.61	2
W8x18	54837	485.18	.61	2
W8x21	65660	580.94	.61	2
W8x24	75042	663.94	.61	2
W10x12	39321	347.90	.61	2
W10x15	49787	440.49	.61	2
W10x17	58445	517.1	.61	2
W10x19	67826	600.09	.61	2
W10x22	83670	740.54	.61	2
W12x14	53395	472.42	.61	2
W12x16	61692	545.83	.61	2
W12x19	76845	679.90	.61	2

CHANNEL	M_{max}		L_c	
	N-m	in.-k	m	ft
C3x4.1	3636	32.18	0.61	2
C3x5	4099	36.28	.61	2
C4x5.4	6380	56.47	.61	2
C5x6.7	9918	87.78	.61	2
C6x8.2	14485	128.15	.61	2
C7x9.8	20107	177.90	.61	2
C8x11.5	26920	238.17	.61	2
C9x13.4	35055	310.15	.61	2

T (tee)	M_{max}		L_c	
	N-m	in.-k	m	ft
WT3x4.25	1312	11.62	0.61	2
WT4x5	2376	20.98	.61	2
WT5x6	3836	35.70	.61	2
WT6x7	5986	52.96	.61	2
WT6x8	7044	59.69	.61	2
WT6x9.5	7605	67.30	.61	2
WT7x11	9590	84.85	.61	2
WT8x13	13459	119.67	.61	2

TABLE IV. ONE-TIER STRUCTURAL STEEL FRAMES

[R is reaction load carrying capacity based on worst case load on interior frame (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC is no greater than length of one module plus 15 cm (6 in.).]

MEMBER BC

L" m	L" ft	R: N 1112 / k .25	2224 / .50	3336 / .75	4448 / 1.0	8896 / 2	17792 / 4	26688 / 6	35584 / 8	TILT ANGLE θ
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	20°
1.22	4	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	
2.44	8	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	30°
1.22	4	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	
1.83	6	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{3}{16}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	40°
1.22	4	$L1x1x1\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{3}{16}$	
2.44	8	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	50°
1.22	4	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
1.83	6	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L3x3x\frac{3}{16}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	60°
1.22	4	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{3}{16}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	
0.61	2	$L1x1\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	70°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{3}{16}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L3x3x\frac{3}{16}$	
2.44	8	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{1}{4}$	

MEMBER AC

m	ft	1112	2224	3336	4448	8896	17792	26688	35584	TILT ANGLE θ
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$\leq 30°$
1.22	4	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$>30°$ $\leq 70°$
1.22	4	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	

TABLE V. TWO-TIER STRUCTURAL STEEL FRAMES

[R is reaction load carrying capacity based on worst case load on interior frame (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of two modules plus 15 cm (6 in.).]

MEMBER BC

L" m	L" ft	R: N 1112 / k .25	R: N 2224 / k .50	R: N 3336 / k .75	R: N 4448 / k 1.0	TILT ANGLE, θ
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	20°
1.22	4	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	
2.44	8	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	30°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	40°
1.22	4	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
2.44	8	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
0.61	2	$L1x1x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	50°
1.22	4	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
1.83	6	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	$L3x3x\frac{3}{16}$	
2.44	8	$L4x4x\frac{1}{4}$	$L4x4x\frac{1}{4}$	$L4x4x\frac{1}{4}$	$L4x4x\frac{1}{4}$	
0.61	2	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	60°
1.22	4	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
1.83	6	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
2.44	8	W6x9	W6x9	W6x9	W6x9	
0.61	2	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{4}x1\frac{1}{4}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	70°
1.22	4	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	
1.83	6	$L4x4x\frac{1}{4}$	$L4x4x\frac{1}{4}$	$L4x4x\frac{1}{4}$	$L4x4x\frac{1}{4}$	
2.44	8	W4x13	W4x13	W4x13	W4x13	

MEMBER AC'

0.61	2	$L3x3x\frac{3}{16}$ C3x4.1	$L3x3x\frac{3}{16}$ C3x4.1	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$ C3x4.1	$L4x4x\frac{1}{4}$ C3x4.1	FOR ALL VALUES OF TILT ANGLE θ $\leq 70^{\circ}$
1.22	4	C3x4.1	C3x4.1	C4x5.4	C5x6.7	
1.83	6	C3x4.1	C5x6.7	C6x8.2	C7x9.8	
2.44	8	C5x6.7 W6x9	C6x8.2 W6x9	C8x11.5 W6x9	C9x13.4 W8x13	

TABLE V. TWO-TIER STRUCTURAL STEEL FRAMES (CONTINUED)

[R is reaction load carrying capacity based on worst case load on interior frame (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of two modules plus 15 cm (6 in.).]

MEMBER BC

R	N	8896	17792	26688	35584	
	k	2	4	6	8	TILT ANGLE θ
L" m	L" ft					
0.61	2	L1x1x$\frac{1}{8}$	L$1\frac{1}{2}$x$1\frac{1}{2}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$	
1.22	4	L$1\frac{1}{2}$x$1\frac{1}{2}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$	20°
1.83	6	L2x2x$\frac{1}{4}$ -----	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ ------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$	
2.44	8	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ ----- -----	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ -------	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ Pipe $2\frac{1}{2}$ STD	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ TS3x3x$\frac{3}{16}$ Pipe $2\frac{1}{2}$ STD	
0.61	2	L1x1x$\frac{1}{4}$	L$1\frac{1}{2}$x$1\frac{1}{2}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$	
1.22	4	L2x2x$\frac{1}{4}$ ----	L2x2x$\frac{1}{4}$ ------	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ -----	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$	30°
1.83	6	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ ------ -------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ Pipe $2\frac{1}{2}$ STD	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ TS3x3x$\frac{3}{16}$ Pipe $2\frac{1}{2}$ STD	
2.44	8	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ -------	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS2x2x$\frac{3}{16}$	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	
0.61	2	L$1\frac{1}{2}$x$1\frac{1}{2}$x$\frac{1}{4}$	L$1\frac{1}{2}$x$1\frac{1}{2}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$	
1.22	4	L2x2x$\frac{1}{4}$ --------	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ ------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$	40°
1.83	6	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L3x3x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS2x2x$\frac{3}{16}$	L$3\frac{1}{2}$x3x$\frac{1}{2}$x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	
2.44	8	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS2x2x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 3 STD TS3x3x$\frac{3}{16}$	
0.61	2	L$1\frac{1}{2}$x$1\frac{1}{2}$x$\frac{1}{4}$	L$1\frac{3}{4}$x$1\frac{3}{4}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$	
1.22	4	L$2\frac{1}{2}$x$2\frac{1}{2}$x$\frac{1}{4}$ ------ ------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ Pipe $2\frac{1}{2}$ STD	50°
1.83	6	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ -------	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS2x2x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	
2.44	8	L4x4x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe $2\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 3 STD TS3x3x$\frac{3}{16}$	W6x12 Pipe 3 STD TS3x3x$\frac{3}{16}$	

TABLE V. TWO–TIER STRUCTURAL STEEL FRAMES (CONCLUDED)

[R is reaction load carrying capacity based on worst case load on interior frame (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of two modules plus 15 cm (6 in.).]

MEMBER BC

L" m	L" ft	R: N 8896 / k 2	R: N 17792 / k 4	R: N 26688 / k 6	R: N 35584 / k 8	TILT ANGLE θ
0.61	2	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{4}$	L1$\frac{3}{4}$x1$\frac{3}{4}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	60°
1.22	4	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ ------ ------	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	60°
1.83	6	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS2x2x$\frac{3}{16}$	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 3 STD TS3x3x$\frac{3}{16}$	60°
2.44	8	W6x9 Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 3 STD TS3x3x$\frac{3}{16}$	W6x12 Pipe 3 STD TS3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{16}$	60°
.61	2	L1$\frac{3}{4}$x1$\frac{3}{4}$x$\frac{1}{4}$	L2x2x$\frac{1}{4}$	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	70°
1.22	4	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ -----	L3x3x$\frac{1}{4}$ TS2x2x$\frac{3}{16}$ ------	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	70°
1.83	6	L4x4x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	L4x4x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$ STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 3 STD TS3x3x$\frac{3}{16}$	W6x9 Pipe 3 STD TS3x3x$\frac{3}{16}$	70°
2.44	8	W4x13 Pipe 3 STD TS3x3x$\frac{3}{16}$	W4x13 Pipe 3 STD TS3x3x$\frac{3}{16}$	W4x13 Pipe 3 STD TS3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{16}$	W4x13 Pipe 3$\frac{1}{2}$ STD TS3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{16}$	70°

MEMBER AC'

L" m	L" ft	R: N / k	8896 / 2	13344 / 3	17792 / 4	22240 / 5	26688 / 6	31136 / 7	35584 / 8	θ
0.61	2	MEMBER SIZE	W6x9 C4x5.4	W6x9 C5x6.7	W6x9 C6x8.2	W6x9 C6x8.2	W6x9 C7x9.8	W8x10 C7x9.8	W8x13 C8x11.5	FOR ALL VALUES OF TILT ANGLE $\theta \leq 70°$
1.22	4	MEMBER SIZE	W6x9 C6x8.2	W8x10 C7x9.8	W8x13 C8x11.5	W8x13 C9x13.4	W12x14 C10x15.3	W12x14 C10x15.3	W12x16.5 C12x20.7	
1.83	6	MEMBER SIZE	W8x10 C9x13.4	W12x14 W10x15.3	W12x16.5 C12x20.7	W12x19 C12x20.7	W12x22 C12x25	W12x22 C12x30	W10x26 C15x33.9	
2.44	8	MEMBER SIZE	W12x19 C12x20.7	W12x22 C12x20.7	W12x22 C12x30	W10x26 C15x33.9	W10x26 C15x33.9	W10x26 C15x33.9	W12x26 C15x40	

TABLE VI. SCHEDULE FOR STRUCTURAL STEEL FOUNDATION CONNECTION AT A (SEE FIGURES 39 TO 44)

[R is reaction load carrying capacity based on worst case load on interior frame (N = Newtons, k = kips).]

R		BASE PLATE TYPE	PLATE THICKNESS		ANCHOR BOLT DIAMETER		ANCHOR BOLT EMBEDMENT	
N	k		mm	in.	mm	in.	cm	in.
		4	12.70	1/2	19.1	3/4	20.3	8
35586	8	5	9.53	3/8	19.1	3/4	12.7	5
		4	12.70	1/2	15.9	5/8	20.3	8
26689	6	5	9.53	3/8	12.7	1/2	12.7	5
		2	12.7	1/2	15.9	5/8	10.2	4
17793	4	4	6.53	1/4	15.9	5/8	20.3	8
		2	9.53	3/8	15.9	5/8	10.2	4
8896	2	4	6.53	1/4	12.7	1/2	15.2	6
		*1	7.94	5/16	12.7	1/2	10.2	4
4448	1	2	6.53	1/4	12.7	1/2	10.2	4
		3	6.53	1/4	12.7	1/2	10.2	4
						1/2		
		*1	6.53	1/4	12.7	1/2	10.2	4
<4448	<1	2	6.53	1/4	12.7	1/2	10.2	4
		3	6.53	1/4	12.7	1/2	10.2	4

* Applicable to one-tier structure only.

TABLE VII. SCHEDULE FOR STRUCTURAL STEEL FOUNDATION CONNECTION AT B (SEE FIGURES 39 to 47)

[R is reaction load carrying capacity based on worst case load on interior frame (N = Newtons, k = kips).]

R		BASE PLATE TYPE	PLATE THICKNESS		ANCHOR BOLT DIAMETER		SHEAR PLATE	ANCHOR BOLT EMBEDMENT	
N	k		mm	in.	mm	in.		cm	in.
35586	8	4	9.53	3/8	25.4	1	N.R.	20.3	8
		7 & 8	12.7 or 9.53*	$\frac{1}{2}$ or $\frac{3}{8}$ *	25.4	1	REQ'D WITH FDN 24a	20.3	8
		6	9.53	3/8	25.4	1	REQ'D WITH FDN 24a	20.3	8
		5	12.70	1/2	25.4	1	N.R.	20.3	8
		2	15.87	5/8	19.1	3/4	N.R.	12.7	5
26689	6	4	9.53	3/8	19.1	3/4	N.R.	20.3	8
		7 & 8	9.53	3/8	19.1	3/4	N.R.	20.3	8
		6	9.53	3/8	19.1	3/4	N.R.	20.3	8
		5	12.70	1/2	19.1	3/4	N.R.	20.3	8
		2	12.70	1/2	15.9	5/8	N.R.	10.2	4
17793	4	4	6.35	1/4	15.9	5/8	N.R.	15.2	6
		7 & 8	9.53	3/8	15.9	5/8	N.R.	15.2	6
		6	6.35	1/4	15.9	5/8	REQ'D WITH FDN 21a	15.2	6
		5	9.53	3/8	15.9	5/8	N.R.	15.2	6
		2	9.53	3/8	12.7	1/2	N.R.	10.2	4
8896	2	4	6.35	1/4	15.9	5/8	N.R.	15.2	6
		7 & 8	6.35	1/4	15.9	5/8	N.R.	15.2	6
		6	6.35	1/4	15.9	5/8	REQ'D WITH FDN 22a	15.2	6
		5	9.53	3/8	12.7	1/2	N.R.	15.2	6
		2	9.53	3/8	12.7	1/2	N.R.	10.2	4
4448	1	3	6.53	1/4	15.9	5/8	N.R.	15.2	6
		2	9.53	3/8	15.9	5/8	N.R.	15.2	6
		** 1	6.35	5/16	12.7	1/2	N.R.	10.2	4
(<4448 AND ≥2224)	(<1 AND ≥0.5)	3	7.94	5/16	12.7	1/2	N.R.	10.2	4
		2	6.35	1/4	12.7	1/2	N.R.	10.2	4
		** 1	7.94	5/16	12.7	1/2	N.R.	10.2	4
<2224	<0.5	3	6.35	1/4	12.7	1/2	N.R.	10.2	4
		2	6.35	1/4	12.7	1/2	N.R.	10.2	4
		** 1	7.94	5/16	12.7	1/2	N.R.	10.2	4

* t = 12.7 mm (1/2 in.) is to be used with 3 in. or 2 in. tube sections or 2-1/2 in. pipe.
t = 9.53 mm (3/8 in.) is to be used with 3-1/2 in. tube section or 3-1/2 in. pipe.

** Applicable to one-tier structures only.

TABLE VIII. TWO-TIER HORIZONTAL BRACING STRUT MEMBER SIZES

[L is span length between support frames.]

Steel

Single Span (See Figures 23 and 28A, 28B, and 28C)

L		m	0.61	1.22	1.83	2.44	3.05 to 4.88	5.18 to 6.10
		ft	2	4	6	8	10 to 16	17 to 20
Compression Member			L1x1x$\frac{1}{8}$	L1$\frac{1}{4}$x1$\frac{1}{4}$x$\frac{1}{8}$	L2x2x$\frac{1}{8}$	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{3}{16}$	W8x10	W8x13

Two, Three, or Four Span (See Figures 24, 29, and 30)

L	m	0.61 to 9.14
	ft	2 to 30
Tension Member		10 mm (3/8 in.) diam. rod

Aluminum

Single Span (See Figures 23 and 28A, 28B, and 28C)

L	m	0.61	1.22	1.83	2.44	3.05	3.66	4.27	4.88	5.18 to 6.10
	ft	2	4	6	8	10	12	14	16	17 to 20
Compression Member		L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$	L2x2x$\frac{1}{8}$	L3x3x$\frac{1}{4}$	L4x4x$\frac{1}{4}$	L5x5x$\frac{3}{8}$	L6x6x$\frac{3}{8}$	W10x7.3	W8x8.32	W8x10

Two, Three, or Four Span (See Figures 23, 29 and 30)

L	m	1.6 to 9.14
	ft	2 to 30
Tension Member		10 mm (3/8 in.) diam. rod

Cold-Formed Steel

Single Span (See Figures 23 and 28A, 28B, and 28C)

L	m	0.61 to 3.66	4.27	4.88 to 6.10
	ft	2 to 12	14	16 to 20
Compression Member		C3x1.16	C3.5x2.14	2C3x2.31

Two, Three, or Four Span (See Figures 23, 29, and 30)

L	m	0.61 to 9.14
	ft	2 to 30
Tension Member		10 mm (3/8 in.) diam. rod

TABLE IX. MAXIMUM MOMENT CAPACITY OF ALUMINUM ALLOY 6061-T6 SHAPES USED AS BEAMS

[M_{max} is maximum moment capacity; L_c is maximum unbraced length of compression member.]

UNEQUAL LEG ANGLES	WT/LENGTH		M_{max}		L_c	
	N/m	lb/ft	N-m	in.-k	m	ft
$L1\frac{3}{4}x1\frac{1}{4}x\frac{1}{8}$	6.13	0.42	211.2	1.87	0.61	2
$L2x1\frac{1}{2}x\frac{1}{8}$	7.29	0.50	273	2.42	.61	2
$L2\frac{1}{2}x2x\frac{1}{8}$	9.48	0.65	316	2.80	.61	2
$L1\frac{3}{4}x1\frac{1}{4}x\frac{1}{4}$	11.8	0.81	397	3.52	.61	2
$L2x1\frac{1}{2}x\frac{1}{4}$	14.0	0.96	524	4.64	.61	2
$L2\frac{1}{2}x1\frac{1}{2}x\frac{1}{4}$	16.2	1.11	900	7.97	.61	2
$L2\frac{1}{2}x2x\frac{1}{4}$	18.4	1.26	1072	9.49	.61	2
$L3x2x\frac{1}{4}$	20.4	1.40	1446	12.8	.61	2
$L3\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	24.5	1.68	1965	17.4	.61	2
$L4x3x\frac{1}{4}$	29.0	1.99	2187	19.36	.61	2
$L3\frac{1}{2}x2\frac{1}{2}x\frac{5}{16}$	30.3	2.08	2632	23.3	.61	2
$L3\frac{1}{2}x3x\frac{5}{16}$	33.2	2.28	2482	21.97	.61	2
$L4x3x\frac{5}{16}$	35.9	2.46	3284	29.07	.61	2
$L4x3x\frac{3}{8}$	42.7	2.93	4236	37.5	.61	2
$L5x3\frac{1}{2}x\frac{5}{16}$	43.9	3.01	4519	39.98	.61	2
$L6x3\frac{1}{2}x\frac{5}{16}$	49.4	3.39	6445	57.05	.61	2
$L6x3\frac{1}{2}x\frac{3}{8}$	58.9	4.04	8831	78.13	.61	2
$L5x3x\frac{1}{2}$	64.2	4.40	8934	79.04	.61	2
$L5x3\frac{1}{2}x\frac{1}{2}$	68.5	4.70	9062	80.18	.61	2
$L6x4x\frac{7}{16}$	71.6	4.91	10029	88.73	.61	2
$L6x3\frac{1}{2}x\frac{1}{2}$	77.5	5.31	12938	114.47	.61	2
$L6x4x\frac{9}{16}$	91.0	6.24	14586	129.05	.61	2
$L6x4x\frac{5}{8}$	100	6.88	16320	144.39	.61	2
$L6x4x\frac{3}{4}$	119	8.15	19288	170.65	.61	2

EQUAL LEG ANGLES	WT/LENGTH		M_{max}		L_c	
	N/m	lb/ft	N-m	in.-k	m	ft
$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	6.26	0.43	154.7	1.37	0.61	2
$L2x2x\frac{1}{8}$	8.31	0.57	222	1.97	.61	2
$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{4}$	11.8	0.81	334	2.96	.61	2
$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{4}$	14.0	0.96	494	4.38	.61	2
$L2x2x\frac{1}{4}$	16.2	1.11	683	6.05	.61	2
$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	20.4	1.40	1066	9.44	.61	2
$L3x3x\frac{1}{4}$	24.5	1.68	1230	10.89	.61	2
$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	29.0	1.99	1484	13.14	.61	2
$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{8}$	29.9	2.05	1672	14.80	.61	2
$L4x4x\frac{1}{4}$	33.2	2.28	1709	15.13	.61	2
$L3\frac{1}{2}x3\frac{1}{2}x\frac{5}{16}$	35.9	2.46	2293	20.3	.61	2
$L3x3x\frac{3}{8}$	36.0	2.47	2387	21.13	.61	2
$L4x4x\frac{5}{16}$	41.3	2.83	2649	23.45	.61	2
$L3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	42.7	2.93	3106	27.50	.61	2
$L3x3x\frac{1}{2}$	47.1	3.23	3208	28.40	.61	2
$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{2}$	55.8	3.83	4529	40.09	.61	2
$L4x4x\frac{7}{16}$	56.9	3.90	4829	42.75	.61	2
$L4x4x\frac{1}{2}$	64.3	4.41	5758	50.97	.61	2
$L5x5x\frac{7}{16}$	71.6	4.91	6383	56.50	.61	2
$L4x4x\frac{9}{16}$	71.7	4.92	6687	59.16	.61	2
$L4x4x\frac{5}{8}$	79.1	5.42	7450	65.91	.61	2
$L5x5x\frac{1}{2}$	81.4	5.58	8285	73.30	.61	2
$L4x4x\frac{3}{4}$	93.4	6.40	8744	77.36	.61	2
$L6x6x\frac{1}{2}$	98.4	6.75	10161	89.90	.61	2

CHANNEL	M_{max}		L_c	
	N-m	in.-k	m	ft
C3x1.42	2720	24.08	0.61	2
C3x1.73	3100	27.44	.61	2
C4x1.85	4996	44.22	.61	2
C4x2.16	5452	48.26	.61	2
C5x2.32	8032	71.1	.61	2
C6x2.83	12066	106.8	.61	2
C6x3.0	12484	110.5	.61	2
C7x3.54	17749	157.1	.61	2
C7x4.23	19712	174.4	.61	2
C8x4.25	24131	213.5	.61	2
C9x4.60	31003	274.3	.61	2
C10x5.28	39892	353.1	.61	2
C10x8.64	53517	473.5	.61	2

WIDE FLANGE	M_{max}		L_c	
	N-m	in.-k	m	ft
W4x4.76	16788	148.6	0.61	2
W5x6.49	30222	267.5	.61	2
W6x4.16	21782	192.8	.61	2
W6x5.4	24663	218.3	.61	2
W6x7.85	46536	411.9	.61	2
W8x5.9	41271	365.3	.61	2
W8x8.32	62308	551.5	.61	2
W8x10.72	77052	682.0	.61	2
W8x11.24	81932	724.9	.61	2
W8x12.99	86848	768.4	.61	2
W10x7.3	62974	557.4	.61	2

T (tee)	M_{max}		L_c	
	N-m	in.-k	m	ft
T2x1.26	649	5.75	.61	2
$T2\frac{1}{4}x1.42$	864	7.65	.61	2
$T2\frac{1}{2}x1.91$	1355	12.0	.61	2
T3x2.72	2476	21.92	.61	2
T4x3.74	4811	42.59	.61	2

TABLE X. ONE-TIER STRUCTURAL ALUMINUM FRAMES

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC is no greater than length of one module plus 15 cm (6 in.).]

MEMBER BC

R	N	1112	2224	3336	4448	TILT ANGLE, θ
	k	.25	.50	.75	1.0	
L" m	L" ft					
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	20°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
2.44	8	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	30°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
2.44	8	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	40°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	50°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	60°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
1.83	6	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
2.44	8	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	70°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
2.44	8	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	

TABLE X. ONE-TIER STRUCTURAL ALUMINUM FRAME (CONTINUED)

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC is no greater than length of one module plus 15 cm (6 in.).]

MEMBER BC

R	N	8896	17792	26688	35584	TILT ANGLE, θ
	k	2	4	6	8	
L" m	L" ft					
.61	2	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	20°
1.22	4	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	20°
1.83	6	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	20°
2.44	8	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	20°
.61	2	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	30°
1.22	4	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	30°
1.83	6	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	30°
2.44	8	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	30°
.61	2	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	40°
1.22	4	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	40°
1.83	6	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3x3x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	40°
2.44	8	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3x3x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	40°
.61	2	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	50°
1.22	4	L2x2x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	50°
1.83	6	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{8}$ Pipe 2$\frac{1}{2}$#5	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3x3x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3x3x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	50°
2.44	8	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3x3x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	L4x4x$\frac{1}{4}$ Pipe 2$\frac{1}{2}$#5	50°

TABLE X. ONE-TIER STRUCTURAL ALUMINUM FRAMES (CONCLUDED)

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC is no greater than length of one module plus 15 cm (6 in.).]

MEMBER BC

R	N	8896	17792	26688	35584	TILT ANGLE, θ
	k	2	4	6	8	
L" m	L" ft					
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$ Pipe $2\frac{1}{2}$#5	$L2x2x\frac{1}{8}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	60°
1.22	4	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3x3x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3x3x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	
2.44	8	$L3x3x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L4x4x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L4x4x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$ Pipe $2\frac{1}{2}$#5	$L2x2x\frac{1}{8}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	70°
1.22	4	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$ Pipe $2\frac{1}{2}$#5	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3x3x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3x3x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$ Pipe 3#5	$L4x4x\frac{1}{4}$ Pipe 3#5	
2.44	8	$L3x3x\frac{1}{4}$ Pipe $2\frac{1}{2}$#5	$L4x4x\frac{1}{4}$ Pipe 3#5	$L4x4x\frac{5}{16}$ Pipe $3\frac{1}{2}$#5	$L4x4x\frac{3}{8}$ Pipe $3\frac{1}{2}$#5	

MEMBER AC

R	N	1112	2224	3336	4448	TILT ANGLE, θ
	k	.25	.50	.75	1.0	
L" m	L" ft					
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$\leq 30°$
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$>30°$ $\leq 70°$
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	

R	N	8896	17792	26688	35584	TILT ANGLE, θ
	k	2	4	6	8	
L" m	L" ft					
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$\leq 30°$
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{3}{4}x1\frac{3}{4}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$>30°$ $\leq 70°$
1.22	4	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L4x4x\frac{1}{4}$	

TABLE XI. TWO-TIER STRUCTURAL ALUMINUM FRAMES

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of two modules plus 15 cm (6 in.).]

L", m	L", ft	MEMBER BC, R: N 1112 / k .25	R: N 2224 / k .50	R: N 3336 / k .75	R: N 4448 / k 1.0	TILT ANGLE, θ
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	20°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
2.44	8	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	30°
1.22	4	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3x3x\frac{1}{4}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	40°
1.22	4	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	50°
1.22	4	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L4x4x\frac{1}{4}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	60°
1.22	4	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L3x3x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
2.44	8	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L4x4x\frac{1}{4}$	
0.61	2	$L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2x2x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	70°
1.22	4	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{8}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	
1.83	6	$L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	
2.44	8	$L3x3x\frac{1}{4}$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	$L4x4x\frac{1}{4}$	$L4x4x\frac{3}{8}$	

TABLE XI. TWO-TIER STRUCTURAL ALUMINUM FRAMES (CONTINUED)

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of two modules plus 15 cm (6 in.).]

MEMBER BC

L", M	L", FT	R: 8896 N (2 k)	R: 17792 N (4 k)	R: 26688 N (6 k)	R: 35584 N (8 k)	TILT ANGLE, θ
0.61	2	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 3#5	L2½x2½x¼ PIPE 3½#5	20°
1.22	4	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 2½#5	L3x3x¼ PIPE 3#5	L3x3x¼ PIPE 4#5	
1.83	6	L2½x2½x¼ PIPE 2½#5	L3½x3½x¼ PIPE 2½#5	L4x4x¼ PIPE 3½#5	L4x4x5/16 PIPE 4#5	
2.44	8	L3½x3½x¼ PIPE 2½#5 ----	L4x4x¼ PIPE 3#5 ----	L4x4x⅜ PIPE 4#5 ----	L5x5x⅜ PIPE 5#5 W6x4.16	
0.61	2	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 3 #5	L2½x2½x¼ PIPE3½#5	30°
1.22	4	L2½x2½x¼ PIPE 2½#5	L3x3x¼ PIPE 2½#5	L3½x3½x¼ PIPE 3#5	L3½x3½x¼ PIPE 4#5	
1.83	6	L3x3x¼ PIPE 2½#5	L4x4x¼ PIPE 3#5	L4x4x⅜ PIPE 3½#5	L4x4x⅜ PIPE 4#5	
2.44	8	L3½x3½x¼ PIPE 3#5 ----	L4x4x⅜ PIPE 3½#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	L5x5x⅜ PIPE 5#5 W6x4.16	
0.61	2	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 3#5	L3x3x¼ PIPE3½#5	40°
1.22	4	L3x3x¼ PIPE 2½#5	L3½x3½x¼ PIPE 2½#5	L4x4x¼ PIPE 3#5	L4x4x5/16 PIPE 4#5	
1.83	6	L3½x3½x¼ PIPE 2½#5 ----	L4x4x⅜ PIPE 3½#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	L5x5x⅜ PIPE 5#5 W6x4.16	
2.44	8	L4x4x¼ PIPE 3#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x⅜ PIPE 5#5 W6x5.4	

MEMBER BC

L", M	L", FT	R: 8896 N (2 k)	R: 17792 N (4 k)	R: 26688 N (6 k)	R: 35584 N (8 k)	TILT ANGLE, θ
0.61	2	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 2½#5	L3x3x¼ PIPE 3#5	L3x3x¼ PIPE 3½#5	50°
1.22	4	L3x3x¼ PIPE 2½#5	L4x4x¼ PIPE 3#5	L4x4x⅜ PIPE 3½#5	L4x4x⅜ PIPE 4#5	
1.83	6	L4x4x¼ PIPE 3#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	L5x5x⅜ PIPE 4#5 W6x4.16	L6x6x⅜ PIPE 5#5 W6x5.4	
2.44	8	L5x5x⅜ PIPE 4#5 W6x4.16	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x7/16 PIPE 5#5 W6x5.4	
0.61	2	L2½x2½x¼ PIPE 2½#5	L2½x2½x¼ PIPE 2½#5	L3x3x¼ PIPE 2½#5	L3½x3½x¼ PIPE3½#5	60°
1.22	4	L3½x3½x¼ PIPE 2½#5 ----	L4x4x¼ PIPE 3#5 ----	L4x4x⅜ PIPE 3½#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	
1.83	6	L4x4x¼ PIPE 3#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x⅜ PIPE 5#5 W6x5.4	
2.44	8	L5x5x⅜ PIPE 4#5 W6x4.16	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x7/16 PIPE 5#5 W6x5.4	L6x6x½ PIPE 6#5 W6x5.4	
0.61	2	L2x2x¼ PIPE 2½#5	L3x3x¼ PIPE 2½#5	L3½x3½x¼ PIPE 3#5	L3½x3½x¼ PIPE 3½#5	70°
1.22	4	L3½x3½x¼ PIPE 2½#5 ----	L4x4x⅜ PIPE 3½#5 ----	L5x5x⅜ PIPE 4#5 W6x4.16	L5x5x⅜ PIPE 4#5 W6x4.16	
1.83	6	L5x5x⅜ PIPE 3½#5 W6x4.16	L5x5x⅜ PIPE 5#5 W6x5.4	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x⅜ PIPE 5#5 W6x5.4	
2.44	8	L5x5x⅜ PIPE 4#5 W6x4.16	L6x6x⅜ PIPE 5#5 W6x5.4	L6x6x⅝ PIPE 6#5 W6x5.4	L8x8x½ PIPE 6#5 W6x7.85	

TABLE XI. TWO-TIER STRUCTURAL ALUMINUM FRAMES (CONCLUDED)

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of two modules plus 15 cm (6 in.).]

MEMBER AC' (FOR ALL VALUES OF TILT ANGLE θ)

R, N		1112	2224	3336	4448	8896	13344
R, k		.25	.50	.75	1.0	2.0	3.0
L", m	L", ft						
0.61	2	C3x1.42 W6x4.16 $L2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	C3x1.42 W6x4.16 $L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{8}$	C4x1.85 W6x4.16 $L3x3x\frac{3}{8}$	C4x2.16 W6x4.16 $L3\frac{1}{2}x3\frac{1}{2}x\frac{3}{8}$	C5x2.32 W6x4.16 —	C6x2.83 W6x4.16 —
1.22	4	C5x2.32 W6x4.16 $L3x3x\frac{3}{8}$	C6x2.83 W6x4.16 $L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{2}$	C7x3.54 W6x4.16 $L4x4x\frac{1}{2}$	C7x3.54 W6x4.16 $L5x5x\frac{1}{2}$	C9x4.6 W6x4.16 —	C10x5.28 W6x5.4 —
1.83	6	C7x3.54 W6x4.16 $L4x4x\frac{1}{2}$	C9x4.6 W6x4.16 $C5x5x\frac{1}{2}$	C10x5.28 W6x4.16 $L5x5x\frac{1}{2}$	C12x7.41 W6x4.16 $L6x6x\frac{1}{2}$	C15x11.71 W6x5.4 —	C15x11.71 W10x7.3 —
2.44	8	C10x5.28 W6x4.16 $L5x5x\frac{1}{2}$	C12x7.41 W6x5.4 $L6x6x\frac{1}{2}$	C12x7.41 W6x5.4 $L6x6x\frac{1}{2}$	C15x11.71 W6x5.4 —	C15x11.71 W10x7.3 —	— W8x10.72 —

R, N		17792	22240	26688	31136	35584
R, k		4	5	6	7	8
L", m	L", ft					
0.61	2	C7x3.54 W6x4.16	C7x3.54 W6x4.16	C8x4.25 W6x4.16	C8x4.25 W6x4.16	C9x4.6 W6x5.4
1.22	4	C12x7.41 W6x5.4	C12x7.41 W8x5.9	C12x7.41 W8x5.9	C15x11.71 W10x7.3	C15x11.71 W10x7.3
1.83	6	C15x11.71 W10x7.3	C15x17.28 W10x7.3	C15x17.28 W8x10.72	PIPE 8#80 W8x10.72	— W8x10.72
2.44	8	— W8x10.72	— W8x10.72	— W8x12.99	— —	— —

TABLE XII. MAXIMUM MOMENT CAPACITY OF COLD-FORMED STEEL SHAPES USED AS BEAMS*

[M_{max} is maximum moment capacity; L_c is maximum unbraced length of compression member; L_c = 0.61 m (2 ft) in all cases.]

TWO CHANNELS BACK TO BACK	M_{max}	
	N-m	in.-k
2C3x2.31	1991	17.63
2C3.5x2.71	2730	24.17
2C4x2.88	3244	28.72
2C5x3.21	3965	35.1
2C6x4.91	5825	51.56
2C7x6.99	8584	75.98
2C6x8.47	13463	119.168
2C7x9.71	17907	158.5
2C8x10.8	22689	200.83
2C9x11.9	28007	247.9
2C10x13.1	30978	274.2
2C10x17.0	43876	388.36
2C12x18.8	56195	497.4

CHANNEL	M_{max}	
	N-m	in.-k
C3x1.16	994	8.83
C3.5x1.36	1214	10.75
C3x1.47	1249	11.06
C3.5x1.68	1681	14.87
C4x1.79	1902	16.84
C3.5x2.14	2096	18.56
C4x2.27	2500	22.13
C5x2.53	2937	25.99
C5x3.5	4507	39.9
C6x4.23	5473	48.45
C6x5.37	8414	74.48
C7x6.17	9171	81.18

HAT SECTION	M_{max}	
	N-m	in.-k
H1.5x.768	237	2.1
H2x.977	425	3.77
H3x1.40	892	7.9
H4x1.81	1450	12.84
H6x2.65	2112	18.7
H8x4.4	3205	28.37
H8x5.57	5339	47.26
H10x11.7	15094	133.6
H10x17.6	28519	252.43

*Notes and figures for structural steel beam splicing are applicable to cold-formed steel.

TABLE XIII. CONNECTION OF TWO CHANNELS BACK TO BACK (SEE SECTION 4.7.1.4)

S max is maximum allowable spacing of bolts; L" is nominal distance between beams when length of member AC' is no greater than length of one module plus 15 cm (6 in.); and Θ is tilt angle.

Member BC

L" (m)	L" (ft)	Tilt angle, Θ 20°	30°	40°	50°	60°	70°	
		Values of S_{max}						
One Tier								
0.61	2	8 3	11 4.5	15 6	18 7	23 9	25 10	cm in.
1.22	4	15 6	23 9	30.5 12	38 15	43 17	51 20	cm in.
1.83	6	23 9	34 13.5	46 18	56 22	66 26	76 30	cm in.
2.44	8	30.5 12	46 18	61 24	76 30	89 35	102 40	cm in.
Two Tier								
0.61	2	18 7	23 9	28 11	34 13.5	40 16	45.5 18	cm in.
1.22	4	35.5 14	46 18	56 22	68.5 27	80 31.5	91 36	cm in.
1.83	6	52 20.5	68.5 27	86 34	103 40.5	119 47	137 54	cm in.
2.44	8	68.5 27	91.5 36	114 45	137 54	160 63	183 72	cm in.

TABLE XIV. ONE-TIER COLD-FORMED STEEL FRAMES

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC is no greater than length of one module plus 15 cm (6 in.).]

MEMBER BC

FOR R ≤ 4448 N (≤1K) USE C3x1.16 IN ALL CASES						
R	N	8896	17792	26688	35584	TILT ANGLE, θ
	k	2	4	6	8	
L"						
m	ft					
0.61	2	C3x1.16	C3x1.16	C3x1.16	C3.5x1.36	20°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3.5x1.36	
1.83	6	C3x1.16	C3x1.16	C3x1.16	C3x1.47	
2.44	8	C3x1.16	C3x1.16	C3.5x1.36	C3x1.47	
0.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.47	30°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3x1.47	
1.83	6	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.68	
2.44	8	C3x1.16	C3x1.16	C3x1.47	C4x1.79	
0.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.47	40°
1.22	4	C3x1.16	C3x1.16	C3.5x1.36	C3x1.47	
1.83	6	C3x1.16	C3x1.16	C3x1.47	C4x1.79	
2.44	8	C3x1.16	C3.5x1.36	C4x1.79	C5x2.0	
0.61	2	C3x1.16	C3x1.16	C3.5x1.36	C3x1.47	50°
1.22	4	C3x1.16	C3x1.16	C3x1.47	C4x1.79	
1.83	6	C3x1.16	C3.5x1.36	C4x1.79	C5x2.0	
2.44	8	C3x1.16	C3x1.61	C5x2.0	2C3x2.31	
0.61	2	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.68	60°
1.22	4	C3x1.16	C3x1.16	C3.5x1.68	C5x2.0	
1.83	6	C3x1.16	C5x1.61	C5x2.0	2C3x2.31	
2.44	8	C3.5x1.36	C4x2.27	2C3x2.31	2C3x2.31	
0.61	2	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.68	70°
1.22	4	C3x1.16	C3.5x1.36	C4x1.79	C4x2.27	
1.83	6	C3.5x1.36	C5x1.61	C4x2.27	2C3x2.31	
2.44	8	C4x2.27	C4x2.27	2C3x2.31	2C3.5x2.71	

MEMBER AC

FOR R ≤ 4448 N (≤1K) USE C3x1.16 IN ALL CASES						
R	N	8896	17792	26688	35584	TILT ANGLE, θ
	k	2	4	6	8	
L"						
m	ft					
0.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	≤30°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3x1.16	
1.83	6	C3x1.16	C3x1.16	C3x1.16	C3x1.16	
2.44	8	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.36	
0.61	2	C3x1.16	C3x1.16	C3x1.16	C3.5x1.36	>30°
1.22	4	C3x1.16	C3x1.16	C3x1.47	C3x1.47	
1.83	6	C3x1.16	C5x1.61	C5x1.61	C5x2.0	
2.44	8	C3x1.16	C4x2.27	C4x2.27	2C3x2.31	

TABLE XV. TWO-TIER COLD-FORMED STEEL FRAMES

[R is reaction load carrying capacity (N = Newtons, k = kips); L" is nominal distance between beams when length of member AC' is no greater than length of one module plus 15 cm (6 in.).]

MEMBER BC

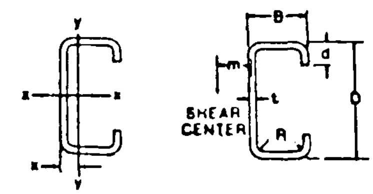
L" m	L" ft	R: N 1112 / k .25	N 2224 / k .50	N 3336 / k .75	N 4448 / k 1.0	N 8896 / k 2	N 17792 / k 4	N 26688 / k 6	N 35584 / k 8	TILT ANGLE, θ
.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.95	2C3x3.65	20°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3.5x3.37	2C3x3.65	
1.83	6	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3.5x3.37	2C3.5x4.27	
2.44	8	C3x1.16	C3x1.16	C3x1.16	C3.5x1.36	2C3.5x2.71	2C3.5x2.71	2C3x3.65	2C3.5x4.27	
.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3x2.95	2C3x3.65	30°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3x2.95	2C3x3.65	
1.83	6	C3x1.16	C3x1.16	C3x1.16	C3.5x1.36	2C3x2.31	2C3.5x2.71	2C3.5x3.37	2C3.5x4.27	
2.44	8	C3x1.16	C3x1.16	C3.5x1.36	C4x1.44	2C3x2.31	2C3.5x2.71	2C3.5x3.37	2C3.5x4.27	
.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3x2.95	2C3.5x3.37	40°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3.5x3.37	2C3x3.65	
1.83	6	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.36	2C3x2.31	2C3.5x2.71	2C3.5x3.37	2C3x3.65	
2.44	8	C3x1.16	C3.5x1.36	C4x1.44	C5x1.61	2C3x2.31	2C3.5x4.27	2C3.5x3.65	2C3.5x5.9	
.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3x2.95	2C3x3.65	50°
1.22	4	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3.5x3.37	2C3.5x4.27	
1.83	6	C3x1.16	C3x1.16	C5x1.61	C5x1.61	2C3x2.31	2C3.5x3.37	2C3.5x4.27	2C3.5x5.9	
2.44	8	C3x1.16	C4x1.44	C5x1.61	C5x2.0	2C3.5x2.71	2C3.5x4.27	2C3.5x5.9	2C3.5x7.45	
.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3.5x2.71	2C3x2.95	60°
1.22	4	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.36	2C3x2.31	2C3x2.31	2C3x2.95	2C3x3.65	
1.83	6	C3x1.16	C3.5x1.36	C3.5x1.44	C5x1.61	2C3x2.31	2C3.5x3.37	2C3.5x4.27	2C3.5x5.9	
2.44	8	C3x1.16	C5x1.61	C5x2.0	C3.5x2.14	2C3.5x2.71	2C3.5x7.45	2C3.5x7.45	2C6x8.47	
.61	2	C3x1.16	C3x1.16	C3x1.16	C3x1.16	2C3x2.31	2C3x2.31	2C3.5x2.71	2C3.5x3.37	70°
1.22	4	C3x1.16	C3x1.16	C3.5x1.36	C3.5x1.36	2C3x2.31	2C3.5x2.71	2C3.5x3.37	2C3.5x4.27	
1.83	6	C3x1.16	C3.5x1.36	C5x1.61	C5x2.0	2C3.5x2.71	2C3.5x4.27	2C3.5x5.9	2C3.5x7.45	
2.44	8	C3x1.16	C5x1.61	C5x2.0	2C3x2.31	2C3.5x3.37	2C3.5x5.9	2C6x8.47	2C9x8.56	

MEMBER AC' (FOR ALL VALUES OF TILT ANGLE θ)

L" m	L" ft	R: N 1112 / k .25	N 2224 / k .50	N 3336 / k .75	N 4448 / k 1.0	N 8896 / k 2.0	N 13344 / k 3.0
.61	2	C3x1.16	C3.5x1.68	2C3.5x2.71	2C4x2.88	2C6x4.91	2C7x5.54
1.22	4	C3.5x1.68	2C4x2.88	2C6x4.91	2C6x4.91	2C8x6.17	2C9x8.56
1.83	6	2C3.5x2.71	2C6x4.91	2C6x4.91	2C7x5.54	2C9x8.56	2C12x14.6
2.44	8	2C4x2.88	2C6x4.91	2C7x5.54	2C8x6.17	2C10x9.34	2C12x14.6

L" m	L" ft	R: N 17792 / k 4	N 22240 / k 5	N 26688 / k 6	N 31136 / k 7	N 35584 / k 8
.61	2	2C8x6.17	2C9x8.56	2C9x8.56	2C10x9.34	2C12x14.6
1.22	4	2C10x9.34	2C12x14.6	2C12x14.6	2C12x14.6	2C12x18.8
1.83	6	2C12x14.6	2C12x14.6	2C12x18.8	-----	-----
2.44	8	2C12x14.6	2C12x18.8	-----	-----	-----

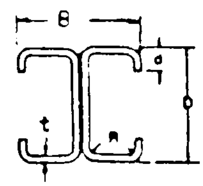
TABLE XVI. SCHEDULE FOR COLD-FORMED STEEL
SINGLE CHANNEL AND DOUBLE CHANNEL SECTIONS



A. Single Channel

SIZE				t		d		R		AREA		WEIGHT	
D		B											
mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm^2	$in.^2$	N/m	lb/ft
178	7.0	69.9	2.75	3.43	.135	20.3	.80	4.76	3/16	1142	1.77	90.0	6.17
152	6.0	63.5	2.50	3.43	.135	17.8	.70	4.76	3/16	994	1.54	78.4	5.37
				2.67	.105	17.8	.70	4.76	3/16	781	1.21	61.7	4.23
127	5.0	50.8	2.00	2.67	.105	17.8	.70	4.76	3/16	645	1.00	51.1	3.50
				1.91	.075	15.2	.60	2.38	3/32	468	.726	36.9	2.53
				1.52	.060	12.7	.50	2.38	3/32	370	.573	29.2	2.00
				1.22	.048	12.7	.50	2.38	3/32	297	.461	23.5	1.61
102	4.0	50.8	2.00	1.91	.075	15.2	.60	2.38	3/32	420	.651	33.1	2.27
				1.52	.060	12.7	.50	2.38	3/32	331	.513	26.1	1.79
				1.22	.048	12.7	.50	2.38	3/32	266	.413	21.0	1.44
89	3.5	50.8	2.00	1.91	.075	15.2	.60	2.38	3/32	395	.613	31.2	2.14
				1.52	.060	12.7	.50	2.38	3/32	312	.483	24.5	1.68
				1.22	.048	12.7	.50	2.38	3/32	251	.389	19.8	1.36
76	3.0	44.5	1.75	1.52	.060	12.7	.50	2.38	3/32	273	.423	21.5	1.47
				1.22	.048	10.2	.40	2.38	3/32	214	.331	16.9	1.16

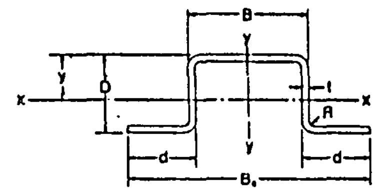
TABLE XVI. SCHEDULE FOR COLD-FORMED STEEL
SINGLE CHANNEL AND DOUBLE CHANNEL SECTIONS (CONCLUDED)



B. Double Channel

SIZE				t		d		R		AREA		WEIGHT	
D		B											
mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm^2	$in.^2$	N/m	lb/ft
305	12.0	178	7.0	3.43	.135	25.4	1.00	4.76	3/16	3490	5.41	274	18.8
				2.67	.105	22.9	.90	4.76	3/16	2703	4.19	213	14.6
254	10.0	178	7.0	3.43	.135	25.4	1.00	4.76	3/16	3142	4.87	248	17.0
				2.67	.105	22.9	.90	4.76	3/16	2432	3.77	191	13.1
				1.91	.075	17.8	.70	2.38	3/32	1729	2.68	136	9.34
229	9.0	165	6.5	2.67	.105	20.3	.80	4.76	3/16	2206	3.42	174	11.9
				1.91	.075	17.8	.70	2.38	3/32	1587	2.46	125	8.56
203	8.0	152	6.0	2.67	.105	20.3	.80	4.76	3/16	2000	3.10	158	10.8
				1.52	.060	15.2	.60	2.38	3/32	1142	1.77	90.0	6.17
178	7.0	140	5.5	2.67	.105	20.3	.80	4.76	3/16	1800	2.79	142	9.71
				1.91	.075	17.8	.70	2.38	3/32	1297	2.01	102	6.99
				1.52	.060	15.2	.60	2.38	3/32	1025	1.59	80.9	5.54
152	6.0	127	5.0	2.67	.105	17.8	.70	4.76	3/16	1568	2.43	124	8.47
				1.52	.060	15.2	.60	2.38	3/32	910	1.41	71.7	4.91
127	5.0	102	4.0	1.22	.048	12.7	.50	2.38	3/32	595	0.922	46.8	3.21
102	4.0	102	4.0	1.22	.048	12.7	.50	2.38	3/32	533	0.826	42.0	2.88
89	3.5	102	4.0	3.43	.135	17.8	.70	4.76	3/16	1381	2.14	109	7.45
				2.67	.105	17.8	.70	4.76	3/16	1090	1.69	86.1	5.90
				1.91	.075	15.2	.60	2.38	3/32	793	1.23	62.3	4.27
				1.52	.060	12.7	.50	2.38	3/32	623	0.966	49.2	3.37
				1.22	.048	12.7	.50	2.38	3/32	502	0.778	39.6	2.71
76	3.0	89	3.5	1.91	.075	12.7	.50	2.38	3/32	677	1.05	53.3	3.65
				1.52	.060	12.7	.50	2.38	3/32	546	0.846	43.1	2.95
				1.22	.048	10.2	.40	2.38	3/32	428	0.663	33.7	2.31

TABLE XVII. SCHEDULE FOR COLD-FORMED STEEL HAT SECTION



SIZE				t		B_0		d		R		WEIGHT	
D		B											
cm	in.	cm	in.	mm	in.	cm	in.	cm	in.	mm	in.	N/m	lb/ft
25.4	10.0	38.1	15.0	3.43	.135	46.0	18.1	4.24	1.67	4.76	3/16	256.8	17.6
25.4	10.0	25.4	10.0	2.67	.105	31.8	12.5	3.40	1.34	4.76	3/16	170.7	11.7
				1.91	.075	29.7	11.7	2.32	.915	2.38	3/32	119.4	8.18
20.3	8.0	10.2	4.0	1.91	.075	14.4	5.68	2.32	.915	2.38	3/32	81.3	5.57
				1.52	.060	13.7	5.38	1.90	.750	2.38	3/32	64.2	4.40
15.2	6.0	7.6	3.0	1.22	.048	10.5	4.14	1.57	.618	2.38	3/32	38.7	2.65
10.2	4.0	5.1	2.0	1.22	.048	7.98	3.14	1.57	.618	2.38	3/32	26.4	1.81
7.6	3.0	3.8	1.5	1.22	.048	6.71	2.64	1.57	.618	2.38	3/32	20.4	1.40
5.1	2.0	2.6	1.0	1.22	.048	5.44	2.14	1.57	.618	2.38	3/32	14.3	0.977
3.8	1.5	1.91	.75	1.22	.048	4.80	1.89	1.57	.618	2.38	3/32	11.2	0.768

TABLE XVIII. SCHEDULE FOR COLD-FORMED STEEL FOUNDATION CONNECTION AT A (SEE FIGURES 39 AND 47)

[R is reaction load carrying capacity (N = Newtons, k = kips).]

R		BASE PLATE TYPE	PLATE THICKNESS		ANCHOR BOLT DIAMETER		ANCHOR BOLT EMBEDMENT	
N	k		mm	in.	mm	in.	cm	in.
35586	8	2	15.87	5/8	19.1	3/4	12.7	5
		5	9.53	3/8	25.4	1	20.3	8
26689	6	2	12.7	1/2	15.9	5/8	10.2	4
		5	9.53	3/8	19.1	3/4	20.3	8
17793	4	2	12.7	1/2	12.7	1/2	10.2	4
		5	6.53	1/4	15.9	5/8	15.2	6
8896	2	2	9.53	3/8	12.7	1/2	10.2	4
		5	6.53	1/4	15.9	5/8	15.2	6
4448	1	1*	7.94	5/16	12.7	1/2	10.2	4
		2	7.94	5/16	12.7	1/2	10.2	4
		4	7.94	5/16	15.9	5/8	15.2	6
<4448	<1	1*	6.53	1/4	12.7	1/2	10.2	4
		2	6.53	1/4	12.7	1/2	10.2	4
		4	6.53	1/4	12.7	1/2	10.2	4

*Applicable to one-tier structure only.

TABLE XIX. SCHEDULE FOR COLD-FORMED STEEL FOUNDATION CONNECTION AT B (SEE FIGURES 39 AND 47)

[R is reaction load carrying capacity (N = Newtons, k = kips).]

R		BASE PLATE TYPE	PLATE THICKNESS		ANCHOR BOLT DIAMETER		ANCHOR BOLT EMBEDMENT	
N	k		mm	in.	mm	in.	cm	in.
35586	8	3	9.53	3/8	19.1	3/4	12.7	5
		5	6.53 (9.53*)	$\frac{1}{4}$ ($\frac{3}{8}$*)	19.1	3/4	20.3 (12.7*)	8 (5*)
26689	6	3	9.53	3/8	12.7	1/2	12.7	5
		5	6.53 (9.53*)	$\frac{1}{4}$ ($\frac{3}{8}$*)	15.9 (12.7*)	$\frac{5}{8}$ ($\frac{1}{2}$*)	20.3 (12.7*)	8 (5*)
17793	4	2	12.7	1/2	15.9	5/8	10.2	4
		*4	12.7	1/2	15.9	5/8	10.2	4
		5	6.53	1/4	15.9	5/8	20.3	8
8896	2	2	9.53	3/8	12.7	1/2	10.2	4
		*4	9.53	3/8	12.7	1/2	10.2	4
		5	6.53	1/4	15.9	5/8	15.2	6
4448	1	*1	7.94	5/16	12.7	1/2	10.2	4
		4	7.94	5/16	12.7	1/2	10.2	4
<4448	<1	*1	6.53	1/4	12.7	1/2	10.2	4
		2	6.53	1/4	12.7	1/2	10.2	4
		4	6.53	1/4	12.7	1/2	10.2	4

*Applicable to one-tier structure only.

TABLE XX. SCHEDULE FOR FOUNDATION TYPE a

[See Figure 36 for dimension designations.]

TYPE	S		W		L		K		T		a+
	m	ft	m	ft	m	ft	m	ft	cm	in.	BARS
1a	.15	.5	1.37	4.5	2.44	8	.46	1.5	40.6	16	6#5
	.30	1.0	1.52	5.0	2.13	7	.61	2.0	40.6	16	6#5
	.46	1.5	1.83	6.0	1.83	6	.61	2.0	40.6	16	6#5
	.61	2.0	1.83	6.0	1.83	6	.61	2.0	40.6	16	6#5
	.76	2.5	2.13	7.0	1.52	5	.76	2.5	40.6	16	6#5
2a	.15	.5	1.22	4	2.74	9	.30	1	40.6	16	6#5
	.30	1.0	1.22	4	2.74	9	.30	1	40.6	16	6#5
	.46	1.5	1.52	5	2.13	7	.38	1.25	40.6	16	6#5
	.61	2.0	1.52	5	2.13	7	.38	1.25	40.6	16	6#5
	.76	2.5	1.83	6	1.83	6	.46	1.50	40.6	16	6#5
3a	.15	.5	1.22	4	2.74	9	.15	.5	30.5	12	4#5
	.30	1.0	1.22	4	2.74	9	.15	.5	30.5	12	4#5
	.46	1.5	1.22	4	2.74	9	.15	.5	30.5	12	4#5
	.61	2.0	1.37	4.5	2.44	8	.30	1	30.5	12	6#5
	.76	2.5	1.52	5	2.13	7	.30	1	30.5	12	6#5
4a	.15	.5	1.37	4.5	2.44	8	.15	.5	30.5	12	4#5
	.30	1.0	1.37	4.5	2.44	8	.15	.5	30.5	12	4#5
	.46	1.5	1.37	4.5	2.44	8	.15	.5	30.5	12	4#5
	.61	2.0	1.37	4.5	2.44	8	.15	.5	30.5	12	6#5
	.76	2.5	1.37	4.5	2.44	8	.15	.5	30.5	12	6#5
5a	.15	.5	1.52	5	1.52	5	.46	1.5	40.6	16	6#5
	.30	1.0	1.83	6	1.22	4	.61	2.0	40.6	16	6#5
	.46	1.5	2.13	7	1.07	3.5	.76	2.5	40.6	16	6#5
	.61	2.0	2.44	8	.91	3	.76	2.5	40.6	16	6#5
	.76	2.5	2.74	9	.91	3	.76	2.5	40.6	16	6#5
6a	.15	.5	1.22	4	1.93	6.33	.30	1	30.5	12	4#5
	.30	1.0	1.52	5	1.52	5	.46	1.5	30.5	12	4#5
	.46	1.5	1.60	5.25	1.52	5	.46	1.5	30.5	12	4#5
	.61	2.0	1.83	6	1.30	4.25	.46	1.5	30.5	12	6#5
	.76	2.5	2.13	7	1.12	3.66	.61	2.0	30.5	12	6#5
7a	.15	.5	1.22	4	1.83	6	.30	1	30.5	12	4#5
	.30	1.0	1.37	4.5	1.74	5.7	.30	1	30.5	12	4#5
	.46	1.5	1.37	4.5	1.74	5.7	.30	1	30.5	12	4#5
	.61	2.0	1.52	5.0	1.52	5	.30	1	30.5	12	4#5
	.76	2.5	1.60	5.25	1.52	5	.30	1	30.5	12	4#5
8a	.15	.5	1.52	5	1.52	5	.15	.5	30.5	12	4#5
	.30	1.0	1.52	5	1.52	5	.15	.5	30.5	12	4#5
	.46	1.5	1.52	5	1.52	5	.15	.5	30.5	12	4#5
	.61	2.0	1.52	5	1.52	5	.15	.5	30.5	12	4#5
	.76	2.5	1.52	5	1.52	5	.15	.5	30.5	12	4#5
9a	.15	.5	1.52	5	1.52	5	-	-	30.5	12	4#5
	.30	1.0	1.52	5	1.52	5	-	-	30.5	12	4#5
	.46	1.5	1.52	5	1.52	5	-	-	30.5	12	4#5
	.61	2.0	1.52	5	1.52	5	-	-	30.5	12	4#5
	.76	2.5	1.52	5	1.52	5	-	-	30.5	12	4#5

+ Re-bar number and size.

TABLE XX. SCHEDULE FOR FOUNDATION TYPE a (CONTINUED)

[See Figure 36 for dimension designations.]

TYPE	S		W		L		K		T		a+
	m	ft	m	ft	m	ft	m	ft	cm	in.	BARS
10a	.15	.5	1.60	5.25	.91	3	.61	2	30.5	12	4#5
	.30	1.0	1.83	6	.81	2.66	.61	2	30.5	12	4#5
	.46	1.5	2.13	7	.70	2.29	.76	2.5	30.5	12	4#5
	.61	2.0	2.44	8	.61	2	.76	2.5	30.5	12	4#5
	.76	2.5	2.74	9	.61	2	.76	2.5	30.5	12	6#5
11a	.15	.5	1.22	4	1.22	4	.30	1	30.5	12	4#5
	.30	1.0	1.52	5	1.00	3.2	.46	1.5	30.5	12	4#5
	.46	1.5	1.68	5.5	.88	2.9	.61	2	30.5	12	4#5
	.61	2.0	1.83	6	.81	2.66	.61	2	30.5	12	4#5
	.76	2.5	2.13	7	.70	2.29	.61	2	30.5	12	4#5
12a	.15	.5	1.22	4	1.22	4	.30	1	30.5	12	4#5
	.30	1.0	1.22	4	1.22	4	.30	1	30.5	12	4#5
	.46	1.5	1.22	4	1.22	4	.30	1	30.5	12	4#5
	.61	2.0	1.52	5	1.00	3.2	.30	1	30.5	12	4#5
	.76	2.5	1.68	5.5	.88	2.9	.46	1.5	30.5	12	4#5
13a	.15	.5	1.22	4	1.22	4	.15	.5	30.5	12	4#5
	.30	1.0	1.22	4	1.22	4	.15	.5	30.5	12	4#5
	.46	1.5	1.22	4	1.22	4	.15	.5	30.5	12	4#5
	.61	2.0	1.22	4	1.22	4	.15	.5	30.5	12	4#5
	.76	2.5	1.22	4	1.22	4	.15	.5	30.5	12	4#5
14a	.15	.5	1.22	4	1.22	4	-	-	30.5	12	4#5
	.30	1.0	1.22	4	1.22	4	-	-	30.5	12	4#5
	.46	1.5	1.22	4	1.22	4	-	-	30.5	12	4#5
	.61	2.0	1.22	4	1.22	4	-	-	30.5	12	4#5
	.76	2.5	1.22	4	1.22	4	-	-	30.5	12	4#5
15a	.15	.5	1.22	4	.69	2.25	.30	1	30.5	12	4#5
	.30	1.0	1.52	5	.55	1.8	.46	1.5	30.5	12	4#5
	.46	1.5	1.83	6	.46	1.5	.46	1.5	30.5	12	4#5
16a	.15	.5	1.07	3.5	.76	2.5	.15	.5	30.5	12	4#5
	.30	1.0	1.22	4	.69	2.25	.30	1	30.5	12	4#5
	.46	1.5	1.52	5	.55	1.8	.30	1	30.5	12	4#5
	.61	2.0	1.52	5	.55	1.8	.30	1	30.5	12	4#5
	.76	2.5	1.83	6	.46	1.5	.46	1.5	30.5	12	4#5
17a	.15	.5	.91	3	.91	3	-	-	30.5	12	4#5
	.30	1.0	.99	3.25	.84	2.76	.15	.5	30.5	12	4#5
	.46	1.5	1.07	3.5	.76	2.5	.15	.5	30.5	12	4#5
	.61	2.0	1.22	4	.69	2.25	.15	.5	30.5	12	4#5
	.76	2.5	1.52	5	.55	1.8	.30	1.0	30.5	12	4#5

+ Re-bar number and size.

TABLE XX. SCHEDULE FOR FOUNDATION TYPE a (CONCLUDED)

[See Figure 36 for dimension designations.]

TYPE	S		W		L		K		T		a +
	m	ft	m	ft	m	ft	m	ft	cm	in.	BARS
18a	.15	.5	.91	3	.91	3	-	-	30.5	12	4#5
	.30	1.0	.91	3	.91	3	-	-	30.5	12	4#5
	.46	1.5	.91	3	.91	3	-	-	30.5	12	4#5
	.61	2.0	.99	3.25	.84	2.76	-	-	30.5	12	4#5
	.76	2.5	1.07	3.5	.76	2.5	-	-	30.5	12	4#5
19a	.15-.76	.5-2.5	.91	3	.91	3	-	-	30.5	12	4#5
20a	.15	.5	1.83	6	1.52	5	.76	2.5	40.6	16	4#5
	.30	1	2.13	7	1.37	4.5	.91	3	40.6	16	4#5
	.46	1.5	2.13	7	1.37	4.5	.91	3	40.6	16	6#5
	.61	2	2.44	8	1.22	4	.91	3	40.6	16	6#5
	.76	2.5	2.44	8	1.22	4	.91	3	40.6	16	6#5
21a	.15	.5	2.13	7	1.52	5	1.07	3.5	40.6	16	6#5
	.30	1	2.13	7	1.52	5	1.07	3.5	40.6	16	6#5
	.46	1.5	2.44	8	1.37	4.5	1.07	3.5	40.6	16	6#5
22a	.15	.5	2.74	9	.61	2	1.22	4	30.5	12	4#5
	.30	1	2.74	9	.61	2	1.22	4	30.5	12	4#5
	.46	1.5	2.74	9	.61	2	1.22	4	30.5	12	4#5
23a	.15	.5	2.13	7	1.98	6.5	1.07	3.5	*	*	4#7
	.30	1	2.44	8	1.83	6	1.22	4.0	*	*	4#7
	.46	1.5	2.74	9	1.52	5	1.37	4.5	*	*	4#7
24a	.15	.5	2.44	8	2.26	7.4	1.22	4	*	*	4#8
	.30	1	2.74	9	2.06	6.75	1.37	4.5	*	*	4#8
	.46	1.5	2.74	9	2.06	6.75	1.37	4.5	*	*	4#8

+ Re-bar number and size.
* Pedestal to be 40.6 cm by 40.6 cm (16 in. by 16 in.) square; bottom slab bars in long direction to be #4 at 6 in. O.C.

TABLE XXI. SCHEDULE FOR FOUNDATION TYPE b

[See Figure 37 for dimension designations.]

TYPE	L (m)	L (ft)
1b	2.70	8.86
2b	2.59	8.5
3b	2.26	7.4
4b	2.13	7.0
5b	1.95	6.4
6b	1.68	5.5
7b*	1.37	4.5
8b	1.45	4.75
9b	1.30	4.25
10b	1.14	3.75
11b*	.91	3.0
12b	.76	2.5

Above Grade Selection

c	m	0.15	0.30	0.46	0.61	0.76
	ft	.5	1.0	1.5	2.0	2.5
A	m	2.51	2.62	2.73	2.82	2.91
	ft	8.25	8.60	8.95	9.25	9.55
a	cm	#4@25	#4@25	#4@20	#5@28	#5@23
BARS	in.	#4@10	#4@10	#4@8	#5@11	#5@9

* Foundation dimensions and reinforcement for typical foundations.

TABLE XXII. SCHEDULE FOR FOUNDATION TYPE c

[See Figure 38 for dimension designations.]

TYPE	DIAMETER		c		A		a +
	cm	in.	m	ft	m	ft	BARS
1c	46	18	0.15	0.5	1.83	6.0	4#5
			.30	1.0	1.95	6.4	4#5
			.46	1.5	2.06	6.75	4#5
			.61	2.0	2.13	7.0	4#5
			.76	2.5	2.21	7.25	4#5
2c	46	18	0.15-0.76	0.5-2.5	2.21	7.25	4#5
3c	46	18	0.15-0.76	0.5-2.5	2.90	9.5	4#5
4c	46	18	0.15	0.5	2.59	8.5	4#5
			.30	1.0	2.67	8.75	4#5
			.46	1.5	2.82	9.25	4#5
			.61	2.0	2.90	9.5	4#5
			.76	2.5	2.97	9.75	4#5
5c	61	24	0.15	0.5	1.98	6.5	4#5
			.30	1.0	2.13	7.0	4#5
			.46	1.5	2.21	7.25	4#7
			.61	2.0	2.29	7.5	4#7
			.76	2.5	2.36	7.75	4#7
6c	61	24	0.15	0.5	2.74	9.0	4#7
			.30	1.0	2.82	9.25	4#7
			.46	1.5	2.97	9.75	4#7
			.61	2.0	3.05	10.0	4#7
			.76	2.5	3.12	10.25	4#7

+ Re-bar number and size.

TABLE XXII. SCHEDULE FOR FOUNDATION TYPE c (CONCLUDED)

[See Figure 38 for dimension designations.]

TYPE	DIAMETER		C		A		a +
	cm	in.	m	ft	m	ft	BARS
7c	61	24	0.15-0.76	0.5-2.5	2.67	8.75	4#7
8c	61	24	.15- .76	.5-2.5	3.20	10.5	4#7
9c	61	24	.15- .76	.5-2.5	3.96	13.0	4#7
10c	61	24	.15- .76	.5-2.5	4.88	16.0	4#7
			0.15	0.5	3.12	10.25	4#7
			.30	1.0	3.20	10.5	4#7
11c	61	24	.46	1.5	3.35	11.0	4#7
			.61	2.0	3.43	11.25	4#7
			.76	2.5	3.58	11.75	4#7
12c	61	24	0.15- .76	0.5-2.5	5.18	17.0	4#7
13c	61	24	.15- .76	.5-2.5	6.10	20.0	4#7
14c	71	28	.15- .76	.5-2.5	3.96	13.0	6#7
15c	76	30	.15- .76	.5-2.5	4.65	15.25	8#7
16c	61	24	.15- .76	.5-2.5	3.66	12.0	4#7
17c	46	18	.15- .76	.5-2.5	3.05	10.0	4#5
18c	46	18	.15- .76	.5-2.5	1.75	5.75	4#5
19c	61	24	.15- .46	.5-1.5	2.90	9.5	4#7
20c	61	24	.15- .46	.5-1.5	5.33	17.5	4#7
			0.15	0.5	2.21	7.25	4#5
21c	46	18	.30	1.0	2.29	7.5	4#5
			.46	1.5	2.36	7.75	4#5
22c	46	18	0.15-0.46	0.5-1.5	3.47	11.4	4#5
23c	91	36	.15- .76	.5-2.5	4.88	16.0	6#9
24c	91	36	.15- .76	.5-2.5	6.55	21.5	6#9
			0.15	0.5	3.66	12.0	4#7
25c	61	24	.30	1.0	3.81	12.5	4#7
			.46	1.5	3.89	12.75	4#7
26c	46	18	0.15-0.76	0.5-2.5	1.62	5.33	4#5
27c	46	18	.15- .76	.5-2.5	1.22	4.0	4#5
28c	30	12	.15- .76	.5-2.5	1.30	4.26	4#5
29c	30	12	.15- .76	.5-2.5	0.91	3.0	4#5
30c	30	12	.15- .76	.5-2.5	1.98	6.5	4#5
31c	30	12	.15- .76	.5-2.5	1.75	5.75	4#5
32c	30	12	.15- .76	.5-2.5	1.60	5.25	4#5
33c	46	18	.15- .76	.5-2.5	1.14	3.75	4#5
34c	30	12	.15- .76	.5-2.5	1.52	5.0	4#5
35c	30	12	15- .76	.5-2.5	1.37	4.5	4#5
36c	30	12	.15- .76	.5-2.5	1.68	5.5	4#5
37c	30	12	.15- .76	.5-2.5	1.14	3.75	4#5
38c	30	12	.15- .76	.5-2.5	0.91	3.0	4#5
39c	30	12	.15- .76	.5-2.5	0.84	2.75	4#5
40c	61	24	.15- .76	.5-2.5	1.91	6.25	4#7
41c	61	24	.15- .76	.5-2.5	1.68	5.5	4#7
42c	30	12	.15- .76	.5-2.5	1.45	4.75	4#5
43c	46	18	.15- .76	.5-2.5	1.91	6.25	4#5
44c	46	18	.15- .76	.5-2.5	2.37	7.77	4#5
45c	46	18	.15- .76	.5-2.5	1.45	4.75	4#5
46c	46	18	.15- .76	.5-2.5	1.32	4.33	4#5
47c	46	18	.15- .76	.5-2.5	1.22	4.0	4#5

+ Re-bar number and size.

TABLE XXIII. FOUNDATION TYPE FOR ONE-TIER TRIANGULAR SYSTEMS

[Reaction is reaction load carrying capacity (N = Newtons, k = kips).]

R, N	1112	2224	3336	4448	8896	17793	26689	35586	TILT ANGLE, θ
R, k	.25	.5	.75	1.0	2	4	6	8	
SUPPORT POINT	FOUNDATION TYPE								
A	39c	36c	33c	26c	3c	11a 8c	6a 10c	2a 13c	20°
B	29c	28c	27c	26c	19a 17c	14a 16c	9a 14c	4a* 15c	
A	38c	35c	33c	26c	3c	11b 7c	7b 9c	6b 21a 12c	30°
B	29c	28c	27c	26c	19a 17c	14a 16c	8a 14c	4a 15c	
A	37c	35c	32c	30c	2c	10b 5c	6b 6c	3b 9c	40°
B	29c	28c	27c	26c	17a 17c	12a 16c	7a 14c	4a 15c	
A	37c	34c	31c	30c	1c	9b 4c	5b 6c	2b 11c	50°
B	29c	28c	27c	26c	17a 17c	12a 16c	7a 14c	22a 15c	
A	37c	34c	31c	30c	12b 1c	8b 4c	4b 6c	1b 11c	60°
B	29c	28c	27c	26c	16a 17c	11a 16c	6a 14c	1a 15c	
A	37c	34c	31c	30c	12b 1c	8b 4c	4b 6c	1b 11c	70°
B	29c	28c	27c	26c	15a 17c	10a 16c	5a 14c	20a 15c	

* Shear key not required.

TABLE XXIV. FOUNDATION TYPE FOR TWO-TIER TRIANGULAR SYSTEMS

[R is reaction load carrying capacity (N = Newtons, k = kips).]

R	N	1112	2224	3336	4448	8896	17793	26689	35586	TILT ANGLE, θ
	k	.25	.5	.75	1.0	2	4	6	8	
SUPPORT POINT		FOUNDATION TYPE								
A		36c	45c	43c	41c	11a 8c	2a 13c	23c 23a	24c 24a	20°
B		42c	26c	41c	40c	16c 22a	15c 21a	23c 23a	24c 24a	
A		35c	47c	18c	44c	11a 8c	20c	4a* 15c	23c 23a	30°
B		42c	26c	41c	40c	16c 22a	15c 21a	23c 23a	24c 24a	
A		37c	47c	18c	43c	22c	9c	5a 14c	15c 21a	40°
B		42c	26c	41c	40c	16c 22a	15c 21a	23c 23a	24c 24a	
A		37c	46c	18c	43c	21c	19c	6b 6c	3b 9c	50°
B		42c	26c	41c	40c	16c 22a	15c 21a	23c 23a	24c 24a	
A		28c	45c	18c	43c	21c	19c	1b 11c	25c	60°
B		42c	26c	41c	40c	16c 22a	15c 21a	23c 23a	24c 24a	
A		28c	45c	18c	43c	21c	19c	1b 11c	25c	70°
B		42c	26c	41c	40c	16c 22a	15c 21a	23c 23a	24c 24a	

* Shear key not required.

4.7.2 Structural Design of Pole-Mounted Systems

This section is divided into three parts: the structural design procedure; the design figures for pole-mounted systems; and the design tables for pole-mounted systems.

4.7.2.1 Structural Design Procedure

A. Choose height of pole
The height should take into account shading from nearby objects, and ease and frequency of maintenance.

B. Determine W: Uniform load applied to beams. (See Figure 48.)
For top and bottom framing beams,

$$W_{TB} = P \times L'/2$$

where

W_{TB}	uniform load applied to top and bottom beams
P	combined load determined in Section 4.3.9
L'	height dimension of one module

C. Determine F_R: the maximum resultant force at top of pole

$$F_R = 1.25 \times L \times L''$$

where

L	span length of beams
L''	nominal distance between top and bottom beams
1.25	factor for wind load reversal

D. Determine pole size
Proceed to Table XXV for Steel, Table XXVI for Aluminum, or Tables XXVII, XXVIII, and XXIX for Wood (Douglas fir) poles. Using the pole height (dimension HT in Figure 48) determined in step 1, the calculated value for F_R determined in step C, and the worst case value of θ, i.e., the largest tilt angle that will be used in the application, select a pole which has an F_R larger than the calculated maximum F_R.

E. Determine sections to be used for framing members (See Figures 48 and 49)
For each member described below calculate the maximum moment (M), then proceed to the indicated table and select the beam section which has the smallest value of M_{max} which is larger than the calculated maximum moment.

(a) Top and bottom framing beams

$$M_{TB} = 1.25\ W_{TB}L^2/8$$

where

M_{TB}	maximum moment developed in top and bottom beams
L	span length between side frames
1.25	factor for wind load reversal

Steel: Table XXX A
Aluminum: Table XXXII

(b) Side framing beams

$$M = 1.25\ W_{TB} \times L \times L'/4$$

where

L' nominal distance between top and bottom beams

Steel: Table XXX B
Aluminum: Table XXXIII

(c) Center beam

$$M = 1.25\ W_{TB} \times L^2/2$$

Steel: Table XXXI
Aluminum: Table XXXIV

(d) Angle adjustment member

Use a minimum of $L1\frac{1}{2}x1\frac{1}{2}x\frac{1}{8}$

F. Determine connection details from Figures 48 and 49 and the following:

(a) For Steel: Read specifications given in Section 4.7.1.2
(b) For Aluminum: Read specifications given in Section 4.7.1.3

G. Read the general notes on foundation design given in Section 4.7.1.5.

H. Determine pole-mounted foundation details.

(a) Steel pole: The steel pole may, at the designer's option, be embedded in concrete or welded to a base plate which is bolted to the concrete foundation.

(1) Pole embedded in concrete:
Proceed to Tables XXXV and XXXVI. Using the pole height (dimension HT in Figure 48) determined in step A, the calculated value of F_R determined in step C, and the worst case value of θ, i.e., the largest tilt angle that will be used in the application, select the appropriate concrete footing and pole embedment from the tables.

Note: The foundation's bearing surface shall be placed below the local frost penetration.

(2) Base plate connection to concrete foundation and concrete foundation design:

- Determine M: the maximum moment sustained by the pole

$$M = 1.25\ F_R \sin \theta \times HT$$

where

HT	pole height
1.25	factor for wind reversal
θ	largest tilt angle that will be used in the application

- Proceed to Table XXXIX and Figures 50 and 51 for base plate and anchor bolt details.

- Proceed to Table XXXV and Figures 48 and 50 for foundation design details.

(b) Aluminum pole: Use procedure in step H(a)(2) above.

(c) Wood pole: The wood pole may be embedded in a concrete footing or directly in the soil. Due to loading considerations, however, embedment of the pole in soil is not always possible. Table XXVII may be used to determine when pole embedment in earth is possible.

(1) For wood pole embedded in concrete:
Proceed to Table XXXV to determine the depth of concrete footing required and Table XXXVII to determine the depth of embedment of the pole into the concrete footing.

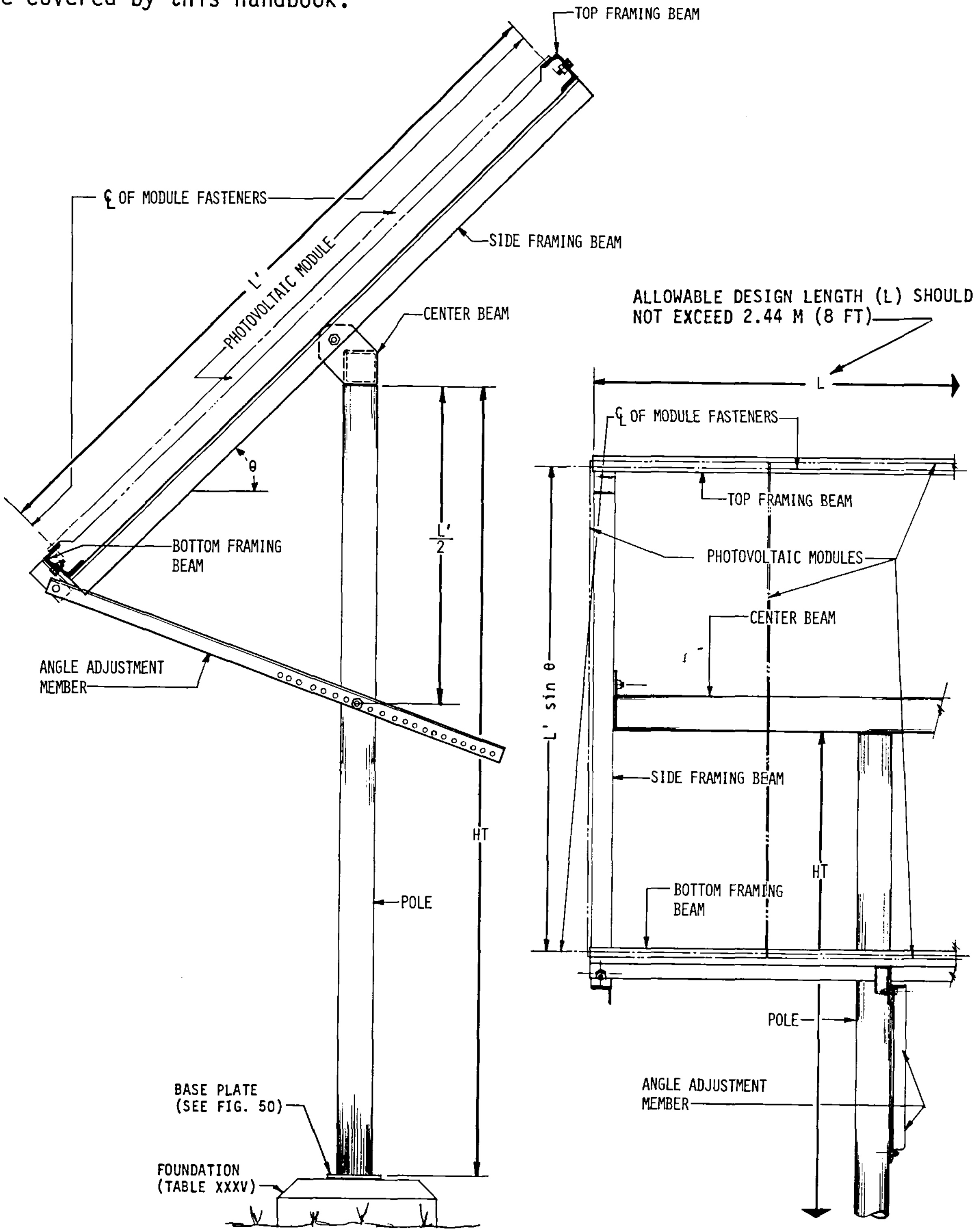
(2) For wood pole embedded into earth:
Proceed to Table XXXVIII to determine the depth of embedment of the pole in earth. Backfill soil may be well tamped native soil, sand, or gravel.

I. Determine shading

If more than one pole-mounted structure is to be used at a single application site, the structures should be located so as to avoid shading. The method given in Section 3.5.1 may be used to determine spacing to avoid shading.

4.7.2.2 Design Figures for Pole-Mounted Systems

This section consists of figures used to design pole-mounted systems of the type covered by this handbook.



A. Side View B. Front View

Figure 48. Pole-Mounted Structural System

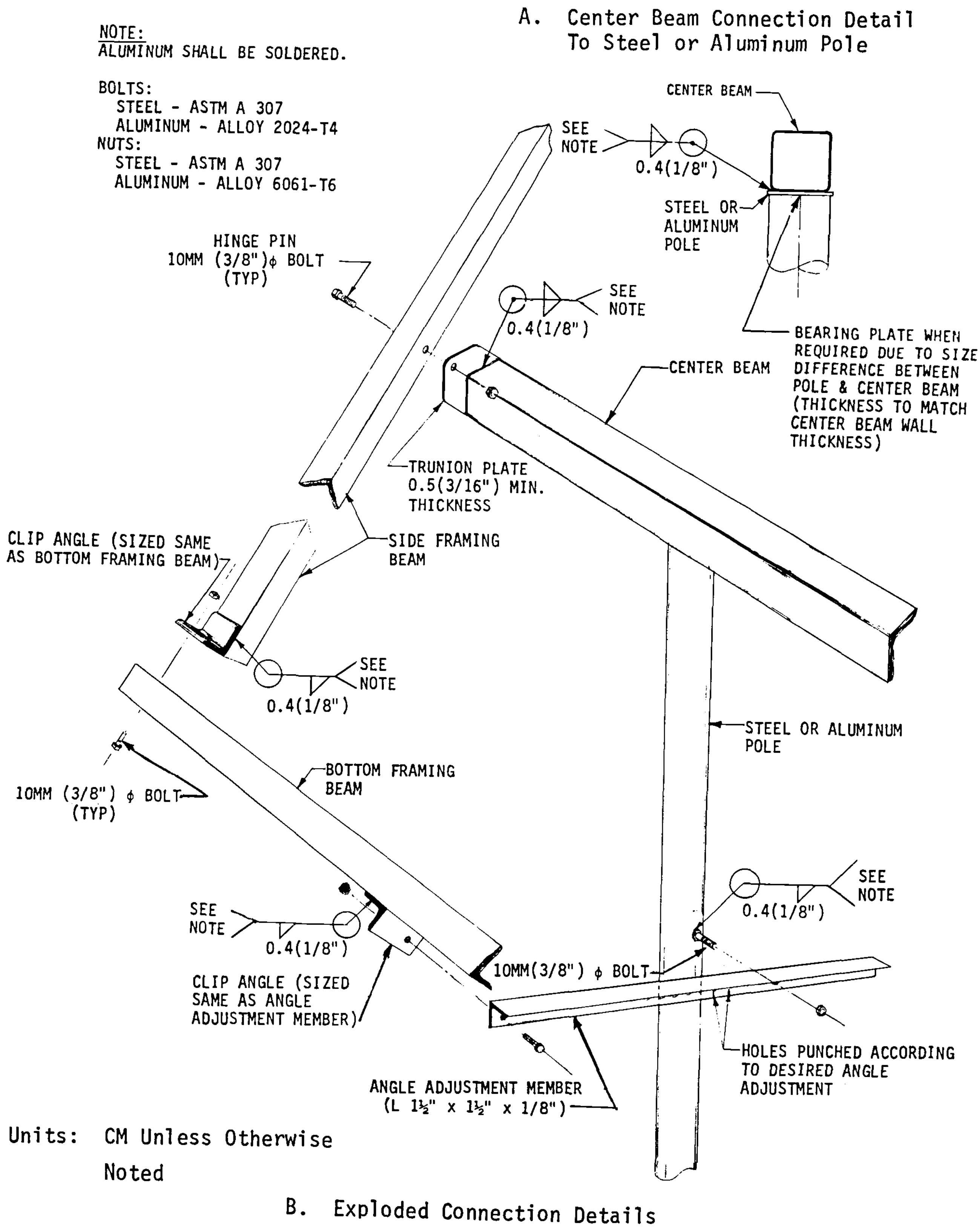
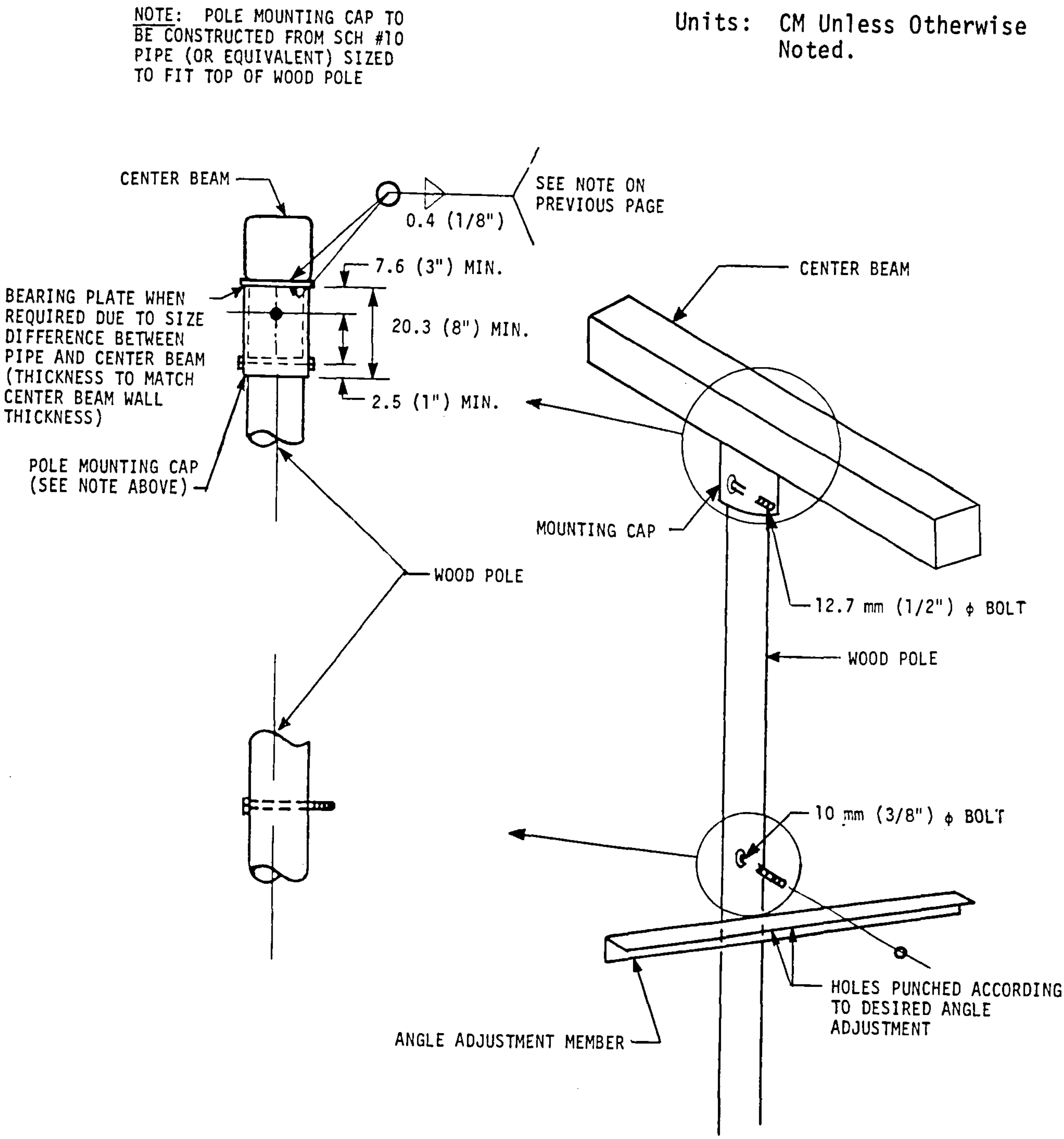


Figure 49. Pole-Mounted Connection Details

Units: CM Unless Otherwise Noted.



C. Connection Details for Wood Pole

Figure 49. Pole-Mounted Connection Details (Concluded)

NOTE: Aluminum shall be soldered.

Units: CM Unless Otherwise Noted

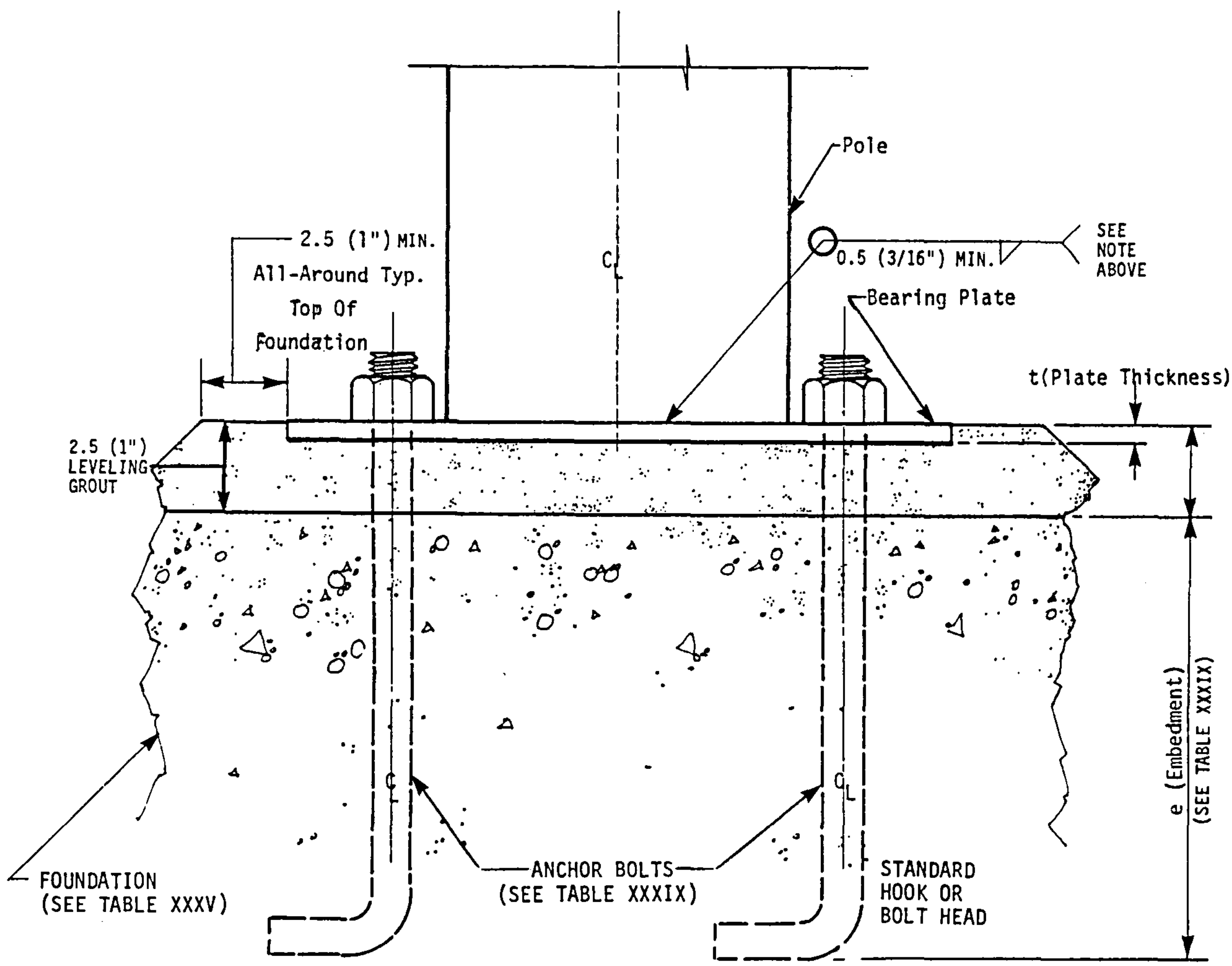
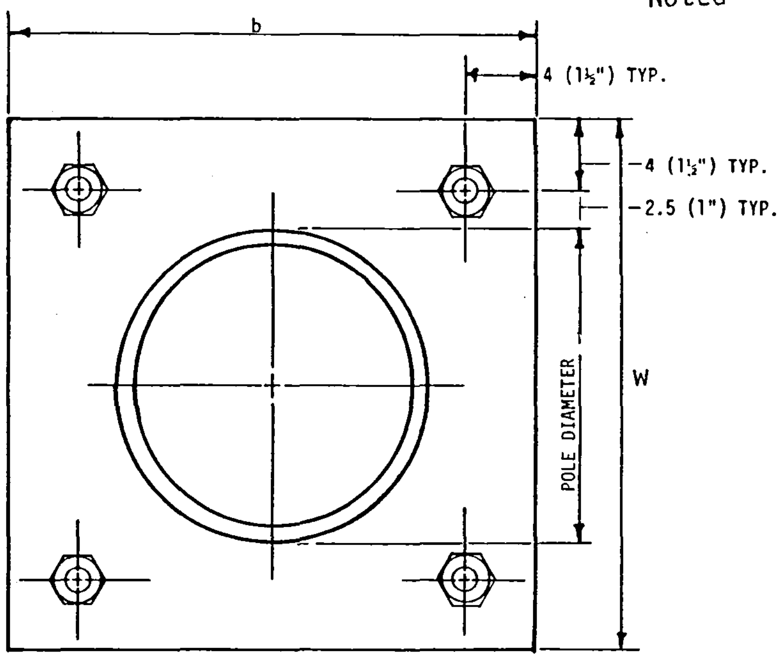
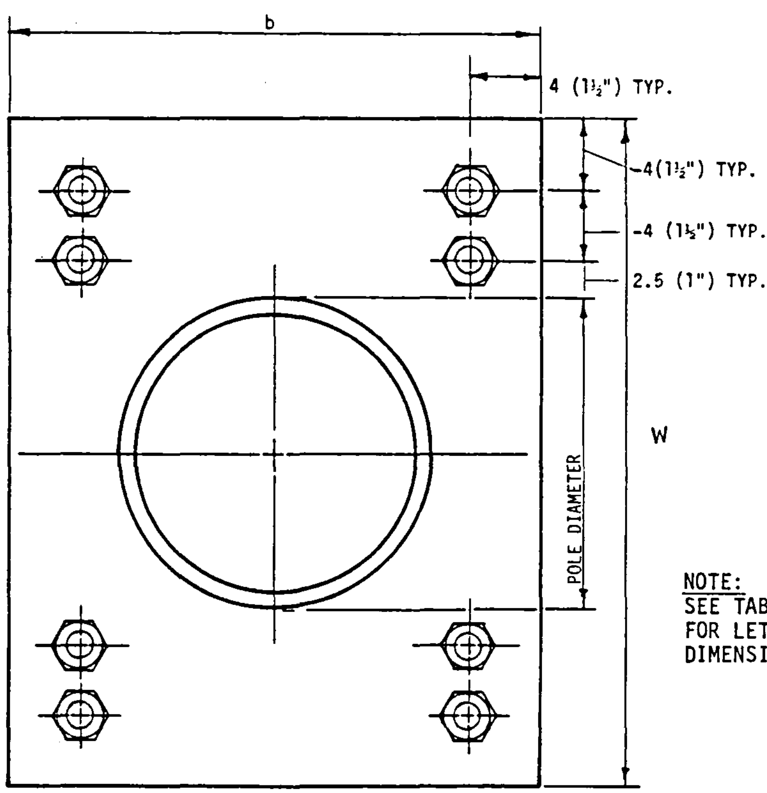


Figure 50. Pole-Mounted Foundation Details

Units: CM Unless Otherwise Noted





NOTE:
SEE TABLE XXXIX
FOR LETTERED
DIMENSIONS

Figure 51. Pole-Mounted Base Plate And Anchor Bolt Details

4.7.2.3 Design Tables for Pole-Mounted Systems

This section consists of tables used to design pole-mountea systems of the type covered by this handbook.

TABLE XXV. STEEL POLES FOR POLE-MOUNTED STRUCTURES

[F_R is maximum resultant force at tope of pole; HT is height of pole.]

F_R / HT	N	445	890	1 134	1779	2224	2669	3114	3558	4003	4448	6672	8896	TILT ANGLE °
	LB	100	200	300	400	500	600	700	800	900	1000	1500	2000	
m	ft													
0.91	3	PIPE $\frac{3}{4}$STD	PIPE 1STD	PIPE 1$\frac{1}{4}$STD	PIPE 1$\frac{1}{4}$STD	PIPE 1$\frac{1}{2}$STD	PIPE 1$\frac{1}{2}$STD	PIPE 2STD	PIPE 2STD	PIPE 2STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	20°
1.83	6	PIPE 1STD'	PIPE 1$\frac{1}{4}$STD	PIPE 1$\frac{1}{2}$STD	PIPE 2STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	
2.74	9	PIPE 1$\frac{1}{4}$STD	PIPE 1$\frac{1}{2}$STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	
3.66	12	PIPE 1$\frac{1}{4}$STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 5STD	
0.91	3	PIPE 1STD	PIPE 1$\frac{1}{4}$STD	PIPE 1$\frac{1}{2}$STD	PIPE 2STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	45°
1.83	6	PIPE 1$\frac{1}{4}$STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 5STD	
2.74	9	PIPE 1$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 4STD	PIPE 4STD	PIPE 5STD	PIPE 6STD	
3.66	12	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 4STD	PIPE 5STD	PIPE 5STD	PIPE 6STD	PIPE 6STD	
0.91	3	PIPE 1STD	PIPE 1$\frac{1}{2}$STD	PIPE 2STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	70°
1.83	6	PIPE 1$\frac{1}{2}$STD	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 5STD	PIPE 5STD	
2.74	9	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 4STD	PIPE 5STD	PIPE 5STD	PIPE 6STD	PIPE 6STD	
3.66	12	PIPE 2STD	PIPE 2$\frac{1}{2}$STD	PIPE 3STD	PIPE 3$\frac{1}{2}$STD	PIPE 4STD	PIPE 4STD	PIPE 5STD	PIPE 5STD	PIPE 5STD	PIPE 5STD	PIPE 6STD	PIPE 8STD	

TABLE XXVI. ALUMINUM POLES FOR POLE-MOUNTED STRUCTURES

[F_R is maximum resultant force at top of pole; HT is height of pole.]

HT, m	HT, ft	F_R, N: 445	890	1334	1779	2224	2669	3114	3558	4003	4448	6672	8896	TILT ANGLE θ
		F_R, lb: 100	200	300	400	500	600	700	800	900	1000	1500	2000	
0.91	3	PIPE $\frac{3}{4}$#40	PIPE 1#40	PIPE $1\frac{1}{4}$#10	PIPE $1\frac{1}{2}$#10	PIPE $1\frac{1}{2}$#40	PIPE 2#10	PIPE 2#40	PIPE 2#40	PIPE 2#40	PIPE $2\frac{1}{2}$#10	PIPE $2\frac{1}{2}$#40	PIPE 3#40	20°
1.83	6	PIPE 1#40	PIPE $1\frac{1}{2}$#10	PIPE 2#10	PIPE 2#40	PIPE $2\frac{1}{2}$#10	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	
2.74	9	PIPE $1\frac{1}{4}$#10	PIPE 2#10	PIPE 2#40	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 5#40	
3.66	12	PIPE $1\frac{1}{4}$#40	PIPE 2#40	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	
0.91	3	PIPE 1#40	PIPE $1\frac{1}{2}$#10	PIPE 2#10	PIPE 2#40	PIPE $2\frac{1}{2}$#10	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	45°
1.83	6	PIPE $1\frac{1}{2}$#10	PIPE 2#40	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE $3\frac{1}{2}$#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	
2.74	9	PIPE 2#10	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	PIPE 6#40	PIPE 6#40	
3.66	12	PIPE 2#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	PIPE 5#40	PIPE 6#40	PIPE 6#40	PIPE 8#20	
0.91	3	PIPE $1\frac{1}{4}$#10	PIPE 2#10	PIPE 2#40	PIPE $2\frac{1}{2}$#10	PIPE $2\frac{1}{2}$#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE 3#40	PIPE 3#40	PIPE 4#40	PIPE 5#40	70°
1.83	6	PIPE 2#10	PIPE $2\frac{1}{2}$#10	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	PIPE 6#40	
2.74	9	PIPE 2#40	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	PIPE 5#40	PIPE 5#40	PIPE 6#40	PIPE 8#20	
3.66	12	PIPE $2\frac{1}{2}$#40	PIPE 3#40	PIPE $3\frac{1}{2}$#40	PIPE 4#40	PIPE 5#40	PIPE 5#40	PIPE 5#40	PIPE 5#40	PIPE 6#40	PIPE 6#40	PIPE 8#20	PIPE 8#30	

TABLE XXVII. CLASS OF WOOD POLES REQUIRED FOR EMBEDMENT IN EARTH (THE CLASS PERTAINS TO ANSI STANDARD POLES (TABLE XXIX))

[F_R is maximum resultant force at top of pole; HT is height of pole.]

HT m	HT ft	F_R N: 445 lb: 100	890 200	1334 300	1779 400	2224 500	2669 600	3114 700	3558 800	4003 900	4448 1000	TILT ANGLE θ
.91	3	10	7	2	-	-	-	-	-	-	-	20°
1.83	6	10	10	9	7	4	1	-	-	-	-	
2.74	9	10	10	10	9	9	7	5	4	3	1	
3.66	12	10	10	10	9	9	6	4	2	-	-	
.91	3	7	-	-	-	-	N/A					45°
1.83	6	10	6	1	-	-						
2.74	9	10	10	7	4	1						
3.66	12	10	9	6	2	-						
.91	3	3	-	-	-	-						70°
1.83	6	10	3	-	-	-						
2.74	9	10	9	4	-	-						
3.66	12	10	7	2	-	-						

TABLE XXVIII. CLASS OF WOOD POLES REQUIRED FOR EMBEDMENT IN CONCRETE (THE CLASS PERTAINS TO ANSI STANDARD POLES (TABLE XXIX))

[F_R is maximum resultant force at top of pole; HT is height of pole.]

HT m	HT ft	F_R N: 445 lb: 100	890 200	1334 300	1779 400	2224 500	2669 600	3114 700	3558 800	4003 900	4448 1000	6672 1500	8896 2000	TILT ANGLE θ
.91	3	10	10	10	10	10	10	10	10	10	10	9	9	20°
1.83	6	10	10	10	10	10	10	10	10	9	9	9	7	
2.74	9	10	10	10	10	9	9	9	9	9	9	7	5	
3.66	12	10	10	10	9	9	9	9	7	7	7	5	4	
.91	3	10	10	10	10	10	10	10	9	9	9	7	7	45°
1.83	6	10	10	10	9	9	9	9	7	7	7	5	4	
2.74	9	10	10	9	9	9	7	7	6	6	5	4	3	
3.66	12	10	9	9	7	7	6	5	5	5	4	3	1	
.91	3	10	10	10	10	10	9	9	9	9	7	7	6	70°
1.83	6	10	10	9	9	9	7	7	7	6	6	4	3	
2.74	9	10	9	9	7	7	6	6	5	5	4	3	1	
3.66	12	10	9	7	7	6	5	4	4	4	3	2	H-1	

TABLE XXIX. WOOD POLE CIRCUMFERENCES
(CLASSES OF WOOD POLES ARE IN ACCORDANCE WITH THE ANSI SPECIFICATIONS FOR WOOD POLES, 05. 1-1974)

[Wood poles shall be treated.]

Class	Minimum Circumference at Top		Minimum Circumference at 1.83 m (6 ft) from Butt	
	m	in.	m	in.
10	0.31	12	0.35	14
9	0.38	15	0.44	17.5
7	0.38	15	0.49	19.5
6	0.43	17	0.53	21.0
5	0.48	19	0.58	23.0
4	0.53	21	0.63	25.0
3	0.58	23	0.68	27.0
2	0.63	25	0.73	29.0
1	0.68	27	0.78	31.0
H-1	0.73	29	1.05	41.5

TABLE XXXA. MAXIMUM MOMENT CAPACITY OF STEEL ANGLES USED AS TOP (AND BOTTOM) FRAMING BEAMS

[M_{max} is maximum moment capacity.]

UNEQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}		EQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}	
	N/m	lb/ft	N-m	in.-k		N/m	lb/ft	N-m	in.-k
$L2\frac{1}{2}x2x\frac{3}{16}$	40.1	2.75	967	8.56	$L2x2x\frac{1}{8}$	24.1	1.65	433	3.83
$L3x2x\frac{3}{16}$	44.7	3.07	1372	12.14	$L1\frac{3}{4}x1\frac{3}{4}x\frac{3}{16}$	30.9	2.12	475	4.20
$L3x2\frac{1}{2}x\frac{3}{16}$	49.4	3.39	1422	12.58	$L2x2x\frac{3}{16}$	35.6	2.44	627	5.55
$L2\frac{1}{2}x1\frac{1}{2}x\frac{5}{16}$	58.4	3.92	1467	12.98	$L2\frac{1}{2}x2\frac{1}{2}x\frac{3}{16}$	44.8	3.07	1000	8.85
$L3x2x\frac{1}{4}$	59.8	4.1	1796	15.89	$L3x3x\frac{3}{16}$	54.1	3.71	1458	12.90
$L3x2\frac{1}{2}x\frac{1}{4}$	65.6	4.5	1855	16.41	$L3x3x\frac{1}{4}$	71.5	4.9	1907	16.87
$L3\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$	71.4	4.9	2497	22.09	$L3\frac{1}{2}x3\frac{1}{2}x\frac{1}{4}$	84.6	5.8	2624	23.22
$L3\frac{1}{2}x3x\frac{1}{4}$	78.7	5.4	2566	22.7	$L4x4x\frac{1}{4}$	96.2	6.6	3470	30.70
$L4x3x\frac{1}{4}$	84.6	5.8	3307	29.26	$L4x4x\frac{3}{16}$	119	8.2	4261	37.70
$L4x3\frac{1}{2}x\frac{1}{4}$	90.4	6.2	3405	30.13					
$L5x3x\frac{1}{4}$	96.2	6.6	5059	44.76					

TABLE XXXB. STEEL ANGLE AND CHANNEL USED AS SIDE FRAMING BEAMS

For $L \leq 1.22$ m (4 ft) use $L2\frac{1}{2}x2x\frac{3}{16}$

For $1.22 \text{ m} < L \leq 2.44$ m ($4 \text{ ft} < L \leq 8$ ft) use C3x4.1

TABLE XXXI. MAXIMUM MOMENT CAPACITY OF STEEL TUBES USED AS CENTER BEAM

[M_{max} is maximum moment capacity.]

TUBE SECTION	M_{max}	
	N-m	in.-k
TS2x2x.25	2756	24.4
TS3x3x.25	6936	61.4
$TS3\frac{1}{2}x3\frac{1}{2}x.25$	9998	88.5
TS4x4x.5	20550	181.9

TABLE XXXII. MAXIMUM MOMENT CAPACITY OF ALUMINUM ANGLES USED AS TOP (AND BOTTOM) FRAMING BEAMS*

[M_{max} is maximum moment capacity.]

UNEQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}		EQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}	
	N/m	lb/ft	N-m	in.-k		N/m	lb/ft	N-m	in.-k
L1$\frac{3}{4}$x1$\frac{1}{4}$x$\frac{1}{8}$	6.13	0.42	211	1.87	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$	6.20	0.43	154	1.37
L2x1$\frac{1}{2}$x$\frac{1}{8}$	7.29	0.50	273	2.42	L2x2x$\frac{1}{8}$	8.31	0.57	222	1.97
L2$\frac{1}{2}$x2x$\frac{1}{8}$	9.48	0.65	316	2.80	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{4}$	11.8	0.81	334	2.96
L1$\frac{3}{4}$x1$\frac{1}{4}$x$\frac{1}{4}$	11.8	0.81	397	3.52	L1$\frac{3}{4}$x1$\frac{3}{4}$x$\frac{1}{4}$	14.0	0.96	494	4.38
L2x1$\frac{1}{2}$x$\frac{1}{4}$	14.01	0.96	524	4.64	L2x2x$\frac{1}{4}$	16.2	1.11	683	6.05
L2$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$	16.2	1.11	900	7.97	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	20.4	1.40	1066	9.44
L2$\frac{1}{2}$x2x$\frac{1}{4}$	18.4	1.26	1072	9.49	L3x3x$\frac{1}{4}$	24.5	1.68	1230	10.89
L3x2x$\frac{1}{4}$	20.4	1.40	1446	12.8	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{4}$	29.0	1.99	1484	13.14
L3$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	24.5	1.68	1965	17.4	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{3}{8}$	29.9	2.05	1672	14.80
L4x3x$\frac{1}{4}$	29.0	1.99	2187	19.36	L4x4x$\frac{1}{4}$	33.2	2.28	1709	15.13
L3$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{5}{16}$	30.3	2.08	2632	23.3	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{5}{16}$	35.9	2.46	2293	20.3
L3$\frac{1}{2}$x3x$\frac{5}{16}$	33.2	2.28	2842	25.15	L3x3x$\frac{3}{8}$	36.0	2.47	2476	21.91
L4x3x$\frac{5}{16}$	35.9	2.46	3284	29.07	L4x4x$\frac{5}{16}$	41.3	2.83	2649	23.45
L4x3x$\frac{3}{8}$	42.7	2.93	4236	37.5	L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{3}{8}$	42.7	2.93	3106	27.5
					L3x3x$\frac{1}{2}$	47.1	3.23	3208	28.4
					L3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{1}{2}$	55.8	3.83	4591	40.62

*See TABLE IX.

TABLE XXXIII. MAXIMUM MOMENT CAPACITY OF ALUMINUM ANGLES USED AS SIDE FRAMING BEAMS*

[M_{max} is maximum moment capacity.]

FOR L = 1.83M (6 FT)

UNEQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}		EQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}	
	N/m	lb/ft	N-m	in.-k		N/m	lb/ft	N-m	in.-k
L1$\frac{3}{4}$x1$\frac{1}{4}$x$\frac{1}{8}$	6.13	0.42	117	1.04	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$	6.26	0.43	139	1.23
L2x1$\frac{1}{2}$x$\frac{1}{8}$	7.29	0.5	324	2.87	L2x2x$\frac{1}{8}$	8.31	0.57	221	1.96
L2$\frac{1}{2}$x2x$\frac{1}{8}$	9.48	0.65	244	2.16	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{4}$	11.8	0.81	253	2.24
L2x1$\frac{1}{2}$x$\frac{1}{4}$	13.42	0.96	428	3.79	L1$\frac{3}{4}$x1$\frac{3}{4}$x$\frac{1}{4}$	14.0	0.96	415	3.68
L2$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{4}$	16.2	1.11	610	5.4	L2x2x$\frac{1}{4}$	16.2	1.11	594	5.26
L2$\frac{1}{2}$x2x$\frac{1}{4}$	18.4	1.26	1134	10.04	L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	20.4	1.4	1054	9.33

FOR L = 2.44M (8 FT)

UNEQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}		EQUAL LEG ANGLES	WEIGHT/LENGTH		M_{max}	
	N/m	lb/ft	N-m	in.-k		N/m	lb/ft	N-m	in.-k
L1$\frac{3}{4}$x1$\frac{1}{4}$x$\frac{1}{8}$	6.13	0.42	66	0.586	L1$\frac{1}{2}$x1$\frac{1}{2}$x$\frac{1}{8}$	6.26	0.43	78	0.69
L2x1$\frac{1}{2}$x$\frac{1}{8}$	7.29	0.5	132	1.17	L2x2x$\frac{1}{8}$	8.31	0.57	221	1.96
L2$\frac{1}{2}$x2x$\frac{1}{8}$	9.48	0.65	324	2.87	L1$\frac{3}{4}$x1$\frac{3}{4}$x$\frac{1}{4}$	14.0	0.96	276	2.45
L3x2$\frac{1}{2}$x$\frac{1}{4}$	22.4	1.54	1448	12.82	L2x2x$\frac{1}{4}$	16.2	1.11	486	4.3
					L2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	20.4	1.4	938	8.3

*For L = 1.22 m (4 ft) use Table XXXII.

TABLE XXXIV. MAXIMUM CAPACITY OF ALUMINUM TUBES AND PIPES USED AS CENTER BEAM

[M_{max} is maximum moment capacity.]

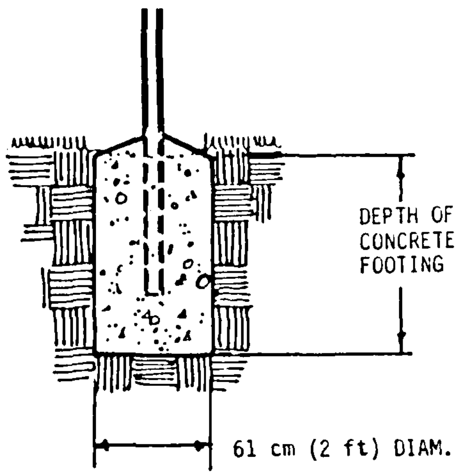
TUBE SECTION	M_{max}	
	N-m	in.-k
TS1.125x1.125x.12	418	3.7
TS1.25x1.25x.12	531	4.7
TS1.375x1.375x.12	700	6.2
TS1.5x1.5x.12	802	7.1
TS1.75x1.75x.156	1386	12.27
TS2x2x.156	1874	16.59
TS2.25x2.25x.156	2435	21.56
TS2.5x2.5x.188	3558	31.5
TS2.75x2.75x.188	4399	38.94
TS3x3x.219	6010	53.2

PIPE	M_{max}	
	N-m	in.-k
PIPE 2$\frac{1}{2}$#80	5027	44.5
PIPE 3#80	8349	73.9
PIPE 3$\frac{1}{2}$#80	8993	79.6
PIPE 4#80	16043	142.0
PIPE 4#120	19455	172.2

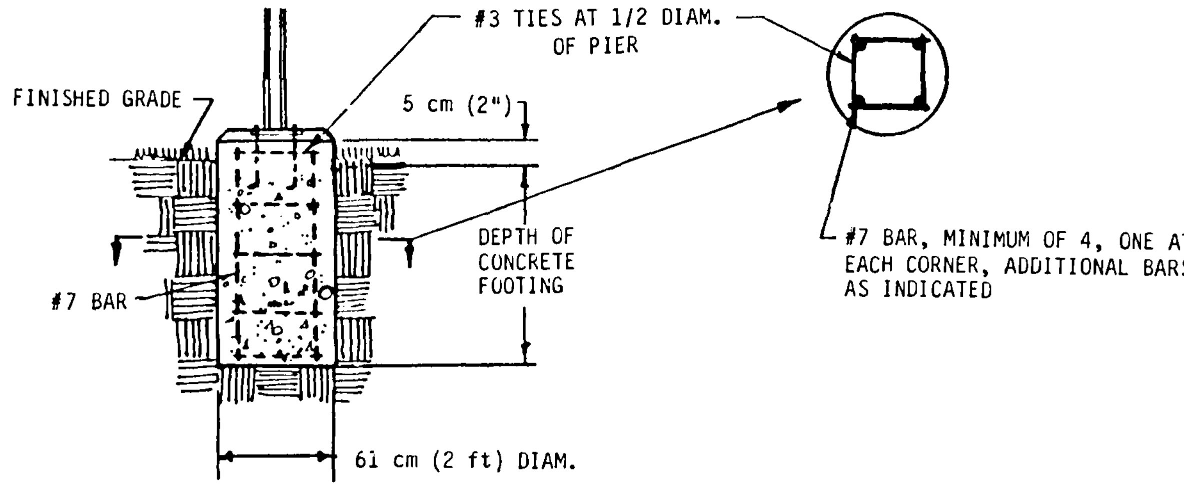
TABLE XXXV. CONCRETE FOOTINGS FOR POLE-MOUNTED STRUCTURES

[F_R is maximum resultant force at top of pole; HT is height of pole]

A. POLE EMBEDDED IN CONCRETE



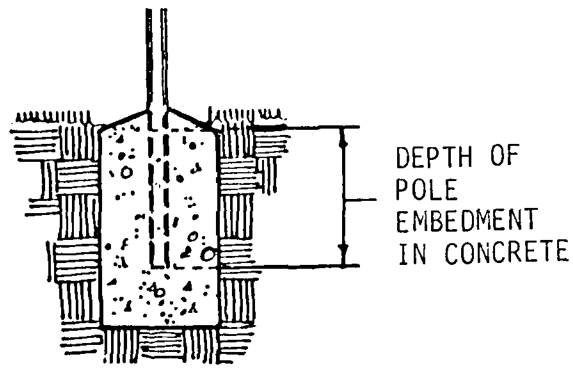
B. BASEPLATE CONNECTION TO CONCRETE FOUNDATION



F_R / HT		N	445		890		1334		1779		2224		2669		3114		3558		4003		4448		6672		8896		TILT ANGLE, θ
		lb	100		200		300		400		500		600		700		800		900		1000		1500		2000		
			DEPTH OF CONCRETE FOOTING																								
m	ft		m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	
0.91	3		0.46	1.5	0.61	2.0	0.67	2.2	0.76	2.5	0.79	2.6	0.88	2.9	0.91	3.0	0.98	3.2	1.01	3.3	1.07	3.5	1.22	4.0	1.37	4.5	
1.83	6		0.55	1.8	0.71	2.3	0.82	2.7	0.91	3.0	0.98	3.2	1.04	3.4	1.10	3.6	1.16	3.8	1.22	4.0	1.25	4.1	1.46	4.8	1.62	5.3	20°
2.74	9		0.64	2.1	0.79	2.6	0.91	3.0	1.00	3.3	1.10	3.6	1.19	3.9	1.25	4.1	1.31	4.3	1.37	4.5	1.40	4.6	1.62	5.3	1.80	5.9	
3.66	12		0.70	2.3	0.88	2.9	1.00	3.3	1.10	3.6	1.19	3.9	1.28	4.2	1.34	4.4	1.40	4.6	1.46	4.8	1.52	5.0	1.77	5.8	1.95	6.4	
0.91	3		0.58	1.9	0.76	2.5	0.88	2.9	0.96	3.2	1.07	3.5	1.13	3.7	1.22	4.0	1.28	4.2	1.34	4.4	1.40	4.6	1.62	5.3	1.83	6.0	
1.83	6		0.73	2.4	0.91	3.0	1.07	3.5	1.19	3.9	1.28	4.2	1.37	4.5	1.43	4.7	1.52	5.0	1.58	5.2	1.65	5.4	1.92	6.3	2.13	7.0	45°
2.74	9		0.82	2.7	1.04	3.4	1.16	3.8	1.31	4.3	1.43	4.7	1.52	5.0	1.62	5.3	1.68	5.5	1.77	5.8	1.83	6.0	2.10	6.9	2.35	7.7	
3.66	12		0.88	2.9	1.13	3.7	1.31	4.3	1.43	4.7	1.55	5.1	1.65	5.4	1.74	5.7	1.83	6.0	1.89	6.2	1.98	6.5	2.29	7.5	2.53	8.3	
0.91	3		0.67	2.2	0.85	2.8	0.98	3.2	1.10	3.6	1.19	3.9	1.25	4.1	1.34	4.4	1.43	4.7	1.49	4.9	1.55	5.1	1.83	6.0	2.04	6.7	
1.83	6		0.79	2.6	1.04	3.4	1.16	3.8	1.31	4.3	1.40	4.6	1.52	5.0	1.58	5.2	1.68	5.5	1.74	5.7	1.83	6.0	2.13	7.0	2.38	7.8	70°
2.74	9		0.88	2.9	1.16	3.8	1.31	4.3	1.46	4.8	1.58	5.2	1.68	5.5	1.77	5.8	1.86	6.1	1.95	6.4	2.01	6.6	2.35	7.7	2.62	8.6	
3.66	12		0.94	3.1	1.28	4.2	1.43	4.7	1.58	5.2	1.71	5.6	1.83	6.0	1.92	6.3	2.01	6.6	2.10	6.9	2.19	7.2	2.53	8.3	2.80	9.2	

TABLE XXXVI. DEPTH OF EMBEDMENT OF STEEL POLE INTO CONCRETE

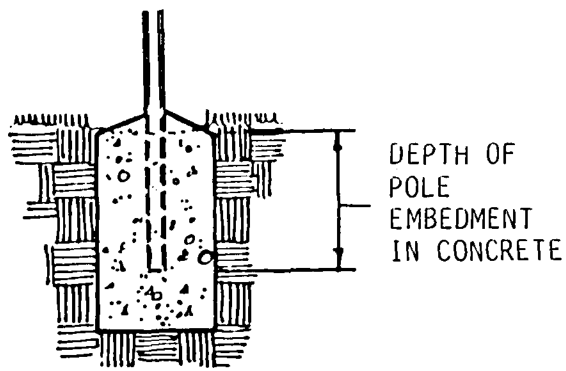
[F_R is maximum resultant force at top of pole; HT is height of pole]



HT, m	HT, ft	F_R 445 N (100 lb): m	ft	890 N (200 lb): m	ft	1334 N (300 lb): m	ft	1779 N (400 lb): m	ft	2224 N (500 lb): m	ft	2669 N (600 lb): m	ft	3114 N (700 lb): m	ft	3558 N (800 lb): m	ft	4003 N (900 lb): m	ft	4448 N (1000 lb): m	ft	6672 N (1500 lb): m	ft	8896 N (2000 lb): m	ft	TILT ANGLE, °
		DEPTH OF EMBEDMENT																								
0.91	3	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	
1.83	6	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	20°
2.74	9	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	
3.66	12	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.30	1.0	
0.91	3	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	
1.83	6	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.24	0.8	0.27	0.9	0.30	1.0	0.30	1.0	45°
2.74	9	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.27	0.9	0.30	1.0	0.30	1.0	0.34	1.1	0.37	1.2	
3.66	12	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.30	1.0	0.30	1.0	0.30	1.0	0.30	1.0	0.37	1.2	0.40	1.3	
0.91	3	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	
1.83	6	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.27	0.9	0.27	0.9	0.30	1.0	0.30	1.0	0.30	1.0	0.37	1.2	70°
2.74	9	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.27	0.9	0.30	1.0	0.30	1.0	0.30	1.0	0.30	1.0	0.30	1.0	0.37	1.2	0.40	1.3	
3.66	12	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.30	1.0	0.30	1.0	0.34	1.1	0.34	1.1	0.37	1.2	0.40	1.3	0.40	1.3	

TABLE XXXVII. DEPTH OF EMBEDMENT OF WOOD POLE INTO CONCRETE

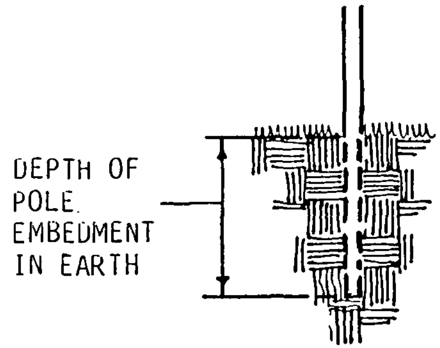
[F_R is maximum resultant force at top of pole; HT is height of pole; θ is tilt angle.]



F_R / HT		N	445		890		1334		1779		2224		2669		3114		3558		4003		4448		6672		8896		TILT ANGLE, θ
		lb	100		200		300		400		500		600		700		800		900		1000		1500		2000		
			DEPTH OF EMBEDMENT																								
m	ft		m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	
0.91	3		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	
1.83	6		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.37	1.2	20°
2.74	9		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.27	0.9	0.30	1.0	0.30	1.0	0.40	1.3	0.46	1.5	
3.66	12		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.27	0.9	0.30	1.0	0.34	1.1	0.40	1.1	0.37	1.2	0.52	1.5	0.52	1.7	
0.91	3		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.37	1.2	
1.83	6		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.30	1.0	0.30	1.0	0.34	1.1	0.37	1.2	0.37	1.2	0.52	1.5	0.52	1.7	45°
2.74	9		0.24	0.8	0.24	0.8	0.24	0.8	0.30	1.0	0.34	1.1	0.37	1.2	0.37	1.2	0.40	1.3	0.43	1.4	0.46	1.5	0.55	1.8	0.64	2.1	
3.66	12		0.24	0.8	0.24	0.8	0.27	0.9	0.34	1.1	0.37	1.2	0.40	1.3	0.43	1.4	0.46	1.5	0.49	1.6	0.52	1.7	0.64	2.1	0.73	2.4	
0.91	3		0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.27	0.9	0.30	1.0	0.3[illegible]	1.2	0.43	1.4	
1.83	6		0.24	0.8	0.24	0.8	0.24	0.8	0.27	0.9	0.34	1.1	0.34	1.1	0.37	1.2	0.40	1.3	0.40	1.3	0.43	1.4	0.52	1.7	0.61	2.0	70°
2.74	9		0.24	0.8	0.24	0.8	0.27	0.9	0.34	1.1	0.37	1.2	0.40	1.3	0.43	1.4	0.46	1.5	0.49	1.6	0.52	1.7	0.64	2.1	0.73	2.4	
3.66	12		0.24	0.8	0.27	0.9	0.34	1.1	0.40	1.3	0.43	1.4	0.46	1.5	0.49	1.6	0.52	1.7	0.55	1.8	0.61	2.0	0.73	2.4	0.85	2.8	

TABLE XXVIII. DEPTH OF EMBEDMENT OF WOOD POLE INTO EARTH

[F_R is maximum resultant force at top of pole; HT is height of pole; θ is tilt angle.]



F_R \ HT		N	445		890		1334		1779		2224		2669		3114		3558		4003		4448		TILT ANGLE, θ
		lb	100		200		300		400		500		600		700		800		900		1000		
			DEPTH OF EMBEDMENT																				
m	ft		m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	m	ft	
0.91	3		1.04	3.4	1.19	3.9	1.22	4.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	20°
1.83	6		1.22	4.0	1.62	5.3	1.71	5.6	1.80	5.9	1183	6.0	1.80	5.9	-	-	-	-	-	-	-	-	
2.74	9		1.37	4.5	1.77	5.8	2.11	6.9	2.10	6.9	2.29	7.5	2.32	7.6	2.35	7.7	2.41	7.9	2.44	8.0	-	7.9	
3.66	12		1.46	4.8	1.92	6.3	2.23	7.3	2.26	7.4	2.44	8.0	2.41	7.9	2.41	7.9	2.41	7.9	-	-	-	-	
0.91	3		1.22	4.0	-	-	-	-	-	-	-	-											45°
1.83	6		1.65	5.4	1.80	5.9	1.83	6.0	-	-	-	-											
2.74	9		1.80	5.9	2.35	7.7	2.35	7.7	2.41	7.9	2.44	8.0											
3.66	12		1.95	6.4	2.29	7.5	2.44	8.0	2.44	8.0	-	-					N/A						
0.91	3		1.22	4.0	-	-	-	-	-	-	-	-											70°
1.83	6		1.86	6.1	1.83	6.0	-	-	-	-	-	-											
2.74	9		2.01	6.6	2.38	7.8	2.41	7.9	-	-	-	-											
3.66	12		2.16	7.1	2.41	7.9	2.44	8.0	-	-	-	-											

TABLE XXXIX. SCHEDULE FOR BASE PLATE AND ANCHOR BOLTS FOR POLE-MOUNTED STRUCTURES

MOMENT		b*		w*		t**		e**		A.B.DIAMETER		# OF BOLTS
N-m	in.-k	cm	in.	cm	in.	cm	in.	cm	in.	cm	in.	
565	5	15.2	6	15.2	6	1.9	3/4	10.16	4	1.27	1/2	4
1129	10	17.8	7	17.8	7	1.9	3/4	10.16	4	1.27	1/2	4
2259	20	17.8	7	17.8	7	1.9	3/4	10.16	4	1.27	1/2	4
3389	30	18.4	7.25	18.4	7.25	1.9	3/4	10.16	4	1.6	5/8	4
4519	40	18.4	7.25	18.4	7.25	1.9	3/4	10.16	4	1.9	3/4	4
5649	50	19.3	7.6	19.3	7.6	1.9	3/4	10.16	4	1.9	3/4	4
6778	60	19.3	7.6	19.3	7.6	1.9	3/4	10.16	4	1.9	3/4	4
7908	70	20.3	8	20.3	8	1.9	3/4	12.7	5	1.9	3/4	4
9038	80	20.3	8	20.3	8	1.9	3/4	12.7	5	2.2	7/8	4
10168	90	22	8.7	22	8.7	1.6	5/8	12.7	5	1.9	3/4	4
11298	100	22	8.7	22	8.7	1.6	5/8	12.7	5	2.2	7/8	4
16947	150	23.9	9.4	23.9	9.4	1.6	5/8	15.24	6	2.2	7/8	4
22595	200	23.9	9.4	31.8	12.5	1.6	5/8	20.32	8	1.9	3/4	8
33893	300	27.4	10.8	44.7	17.6	1.27	1/2	15.24	6	2.2	7/8	8
38413	340	27.4	10.8	50	19.7	1.27	1/2	20.32	8	2.2	7/8	8

* See Figure 50 for designation of dimensions
** See Figure 51 for designation of dimensions

4.8 Installation and Maintenance Factors

A complete installation and maintenance plan should be written in the form of a statement of work which includes:

- A fabrication and installation plan
- A procurement plan
- A quality control plan
- A maintenance plan

A. Statement of Work

The statement of work may be divided into the following tasks:

Task 1. Management and System Integration
Task 2. System Installation
Task 3. System Checkout
Task 4. System Performance Verification
Task 5. Maintenance

(a) Task 1: Management and System Integration

Task 1 involves the following steps:

(1) Planning
(2) Coordinating and administering the entire project work effort
(3) Managing the work of subcontractors and consultants
(4) Maintaining quality control
(5) Defining all work tasks
(6) Monitoring and controlling cost
(7) Scheduling and maintaining schedule control
(8) Managing and controlling change orders
(9) Reporting progress at periodic intervals
(10) Accepting the completed work
(11) Establishing a safety plan (optional)

(b) Task 2. System Installation

Task 2 involves:

(1) Site preparation
- Preparing the necessary documents for procurement of labor, materials and services
- Inspecting the site during construction
- Certifying recommendation or approval of contractor's invoices for payment
- Monitoring all safety matters
- Initiating erosion-control and water drainage procedures for the site (if necessary)
- Complete grading of array site
- Fencing
- Accepting completed work

(2) Array structure and electrical connections
- Installing foundations and structural members, and making appropriate connections
- Installing modules, cabling, switches, and junction boxes on array and making appropriate electrical connections

(3) Safety
- Installating electrical grounding and lightning protection

(c) Task 3. System Checkout

Task 3 includes:

(1) Defining tests required
(2) Writing procedure for system check-out
(3) Inspecting all work
(4) Conducting start-up test of complete system under normal operating conditions

(d) Task 4. System Performance Verification

Task 4 involves:

(1) Writing test plan for a trial operation
(2) Conducting all-up system trial operation
(3) Conducting approved experiments

(e) Task 5. Maintenance Plan

This task involves:

(1) Yearly or bi-yearly visits to site
(2) Inspecting and testing of modules
(3) Inspecting foundation, structure, and mechanical connections
(4) Inspecting electrical connections and grounding
(5) Reporting results of inspection
(6) Repairing/replacing parts and components as necessary

4.9 Cost Analysis

A necessary part of any practical engineering design is the cost analysis. This is especially critical in photovoltaic array design, where the cost for an array for one particular application may vary significantly depending on the module type and construction materials used.

The cost analysis for the array structure may be divided into the following parts:

- Preliminary Costs
- Site Preparation Costs
- Array Structure Costs
- Array Electrical Costs
- Miscellaneous Costs

A. Preliminary Costs

The preliminary costs include:

(a) Building permit fees, as described in Section 4.2
(b) Plan checking fee (not necessarily required: the local zoning board should be consulted)
(c) Site survey

B. Site Preparation Costs

Site preparation costs include the material and labor, including machinery, cost of:

(a) Grading the site
(b) Clearing the site
(c) Roads
(d) Fences
(e) Drainage
(f) Relocation or modification of existing structures
(g) Hole preparation for foundation

C. Array Structure Costs

Array structure costs include the material, labor, and machinery costs for:

(a) Photovoltaic modules
(b) Structural members (beams and columns)
(c) Complete foundation
(d) Mechanical connections (bolts, welds, etc.)

D. Array Electrical Costs

Array electrical costs include the material, labor, and machinery costs for:

(a) Junction boxes, conduits, etc.
(b) Cabling
(c) Grounding
(d) Lightning protection
(e) Mechanical connections for conduit or junction boxes to array structure

E. Miscellaneous Costs

Miscellaneous costs include:

(a) Insurance on materials and labor
(b) Sales tax
(c) Overhead and profit for sub-contractors
(d) Array cleaning or washing equipment
(e) Maintenance costs (cost to make regular checkout of system and cost to replace a module or sets of modules)
(f) Inflation

The cost of each part and subpart of the cost analysis should be calculated as the sum of four distinct costs:

- Cost of equipment or material at location of purchase
- Cost of delivery of equipment or material to site
- Cost of checking material at site (before and after assembly in array)
- Cost of labor for on-site fabrication, assembly, and installation

Once the costs are determined, they should be organized in table form. A sample structural cost estimate table is shown herein. The sub-total costs can then be added to find the overall cost.

In order to adequately perform the cost analysis, the designer must have adequate information sources. These include the following:

- Manufacturers (lumber yards, fabricators, aluminum exteriors)
- Local suppliers and supply houses
- Trade associations for the respective construction materials (See Section 7.0)
- Cost Analysis Manuals
  <u>Means</u>
  <u>Dodge</u>
  <u>Building Cost File (Regional Editions)</u>
- Construction contractors

<u>STRUCTURAL COST ESTIMATE (PRELIMINARY)</u>

	UNIT COSTS			COST		TOTAL
ITEM	QUANTITY	MATERIAL	INSTALLATION	MATERIAL	INSTALLATION	COSTS
Site Preparation						
Concrete Foundation(s)						
Steel						
Sub-Total						
15% Insurance on labor						
6% Sales Tax						
Sub-Total						
20% Overhead and Profit						
Sub-Total						
10% Inflation						
TOTAL						

5.0 ARRAY/STRUCTURAL DESIGN EXAMPLE PROBLEMS

This section illustrates the use of this handbook for designing triangular and pole-mounted array/structural systems for photovoltaic system applications. There are five examples covering the three generic systems discussed in this handbook. Three of the examples (Examples 1, 3, and 5) use English Units, while the other two examples (Examples 2 and 4) use SI Units. The first example is for a 1 kWp dedicated system and is worked out in detail for structural steel, aluminum, and cold-formed steel with a cost analysis for each. (Costs for modules and electrical hardware are not included, however.) The next two examples, one for 200 Wp and one for 4 kWp, are used to show the difference in PV module and structural member sizes when designing different systems. In addition, the third example illustrates the method used to determine the sun's position relative to site location as a function of the time of year and time of day, and the method used to determine row spacing to avoid shading from one row to the next. The fourth example, for a 1 kWp portable system is used to illustrate the difference between dedicated and portable triangular systems. Finally, the fifth example is for a 200 Wp pole-mounted system.

In the cost analysis it should be noted that the costs are valid only to show how to do a cost analysis. Costs of the different materials are a function of location in the world, time when ordered, and amount ordered, and so may vary widely depending upon these restrictions. In addition, the costs for labor and services vary significantly as a function of location and time. Therefore the costs given should not be considered actual costs for building a photovoltaic array/structural system.

5.1 Design Example 1: 1 kWp Dedicated (English Calculations)

A. Description

Site location:	Near Puget Sound, Washington (Latitude = 47.5° N, Longitude = 123° W).
Site conditions:	Level site, no size or shape restrictions on array field, and no shading from nearby trees or structures.
Weather conditions:	Consult tables and figures.
Tilt angle:	Design for permanent (fixed) tilt of 50° (See Step 1, item (b) in Section 2.0).
Array orientation:	Orient array so that it faces due south.

B. Module Determination

13 percent efficient module
Size: 2 ft x 2 ft
Weight: 3 lb/ft^2

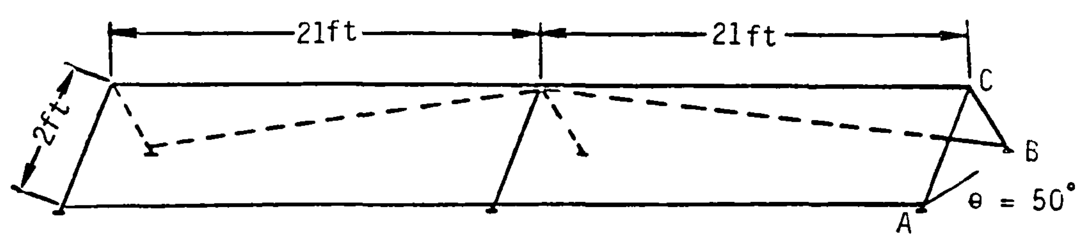
C. Array Size

1 kWp/m^2 insulation and 13 percent efficiency implies that 1 kWp requires 7.7 m^2 or 82.9 ft^2 of array. The specific size of the array, however, must be an integral number of modules. In addition, the integral number of modules needed to produce the desired voltage must be

considered (See Step 1, item (b) in Section 2.0). Since the voltage is not given in this example, the array size is taken as 84 ft^2 (21 modules).

D. Array Layout

The layout of the structure is determined by the site conditions. Since there are no apparent restrictions for this site, a one-tier, two-bay structure was chosen for this example. A small site may require a two-tier structure with smaller spans or a structure consisting of multiple rows. This layout, however, assumes there are 42 ft of clear area within which to place the structure so a one-tier structure will be used. Other considerations for layout are serviceability and maintenance. Generally speaking, a two-tier structure or multiple row structural system is more costly to maintain and more difficult to service. The structural system chosen for this example is illustrated in the following figure:



E. Codes

No codes consulted; design in accordance with this handbook.

F. Loads

(a) Dead Load: The dead load is equal to the weight of the array plus the weight of the structure. The weight of the array is provided by the manufacturer; 3 psf in this case. The weight of the structure may be carefully estimated by section designations or computed using the density of the material. In this example, all three primary materials covered in the handbook (structural steel, aluminum, and cold-formed steel) are considered. Of these, steel is the heaviest. The average weight of the steel structure is 4 psf (this is a conservative estimate). After the structure is designed, the weight can be computed and compared to the estimated value; if necessary, the design may be revised if the weight is underestimated. The dead load, DL = weight of structure + weight of modules = 4 psf + 3 psf = 7 psf.

(b) Live and other loads: The wind map (Figure 9) indicates a 30 psf wind pressure for the site. Design for 25 psf since the structure is less than 30 ft above ground. The seismic load, V = 0.24 DL = 0.24 x 7 psf = 1.68 psf. The snow map (Figure 10) indicates a 10 psf snow pressure for the site. Reduction is not done because snow load is less than 15 psf. Ice loads are not of concern. Maintenance load is 15 psf (See Section 4.3.2).

Using the method given in Section 4.3.9:

Total Load = Dead Load + Live Load = 7 psf + 15 psf = 22 psf.
Total Load = Dead Load + Wind Load = 7 psf + 25 psf = 32 psf.
Total Load = Dead Load + Seismic Load = 7 psf + 1.68 psf = 8.68 psf.
Total Load = Dead Load + Snow Load = 7 psf + 10 psf = 17 psf.

Use total load of 32 psf as the design criterion since this is most critical load.

Note: DL and WL are combined without regard to direction for simplicity - this is a conservative approach.

G. Structural Design

See Section 4.7.1.1, Structural Design Procedure and Figures 11C, 14B, and 16A.

(a) Determine W: uniform load applied to the beams.
For top and bottom beams:

W = Total Load x L'/2

where

L' height of one module

W = 32 lb/ft x 2 ft x 1/2 = 32 lb/ft

(b) Determine M: The maximum moment developed in top and bottom beams.
For two continuous span beams:

$M = 1.25\ WL^2/8$

where

L span length between support frames

$M = 1.25 \times 32\ \text{lb/ft} \times (21\ \text{ft})^2 \times 1/8 \times 12\ \text{in./ft} \times 0.001\ \text{k/lb} = 26.46\ \text{k-in.}$

(c) Determine R: The maximum reaction for the top and bottom beams.
For two continuous span beams:

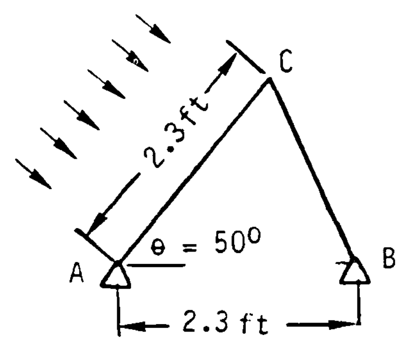
R = WL x 1.25

where

L span length between support frames

R = 32 lb/ft x 21 ft x 1.25 x 0.001 k/lb = 0.84 k

(d) Determine lengths of members AC and BC.



AC = AB = height of module + 3 in. + 1/2 in. = 2.3 ft = L"; tilt angle = 50° up from horizontal.

$$BC = \sqrt{2L''^2 - 2L''^2 \cos \theta}$$

$$BC = \sqrt{2(2.3)^2 - 2(2.3)^2 \cos 50^\circ} = 1.94 \text{ ft}$$

(e) Determine the sections to be used:

(1) Structural Steel

Top and bottom beams (see Table III): Based on the maximum moment (M), the options are: L4x3x1/4 (5.8 lb/ft), L4x4x1/4 (6.6 lb/ft), WT5x6 (6 lb/ft), C3x4.1 (4.1 lb/ft), and W6x9 (9 lb/ft). Selection is based on weight and availability. Assuming equal availability, choose C3x4.1 since it is the lightest section. Recalculation of the equations using the new value of P (combined load) for actual member weight (4.1 lb/ft versus original estimate of 4 lb/ft) results in selection of the same member section, i.e., C3x4.1 for the top and bottom beams.

Members AC and BC: Member selection based on R = 0.84 kips.

For member AC (see Table IV): L1x1x1/8.
For member BC (see Table IV): L1x1x1/8.

(2) Aluminum

Top and bottom beams (see Table IX): Based on the maximum moment (M) calculated for structural steel, the options are: L4x3x5/16 (2.46 lb/ft), L3-1/2x3-1/2x3/8 (2.93 lb/ft), C3x1.73 (1.73 lb/ft), W4x4.76 (4.76 lb/ft), and T4x3.74 (3.74 lb/ft). Assuming equal availability, choose lightest section: C3x1.73. Recalculation of the equations using the new value for P (combined load) for actual member weight (1.73 lb/ft versus orginal estimate of 4 lb/ft) results in selection of the same member section, i.e., C3x1.73 for the top and bottom beams. New values are M = 24.58 k-in. and R = 0.78 kips.

Members AC and BC: Member selection is based on R = 0.78 kips.

For member AC (see Table X): L1-1/2x1-1/2x1/8.
For member BC (see Table X): L1-1/2x1-1/2x1/8.

(3) Cold-Formed Steel

Top and bottom beams (see Table XII): Based on the maximum moment (M) calculated for structural steel, the options are: 2C4x2.88 (2.88 lb/ft), C5x3.5 (3.5 lb/ft) and H8x4.4 (4.4 lb/ft). Assuming equal availability, choose lightest section: 2C4x2.88. Recalculation using the new value for P (combined load) for actual member weight (2.88 lb/ft versus original of 4 lb/ft) results in selection of section: C5x2.53 (2.53 lb/ft) for the top and bottom beams. This member is lighter than the member previously chosen. New values are M = 25.24 k-in. and R = 0.80 kips.

Members AC and BC: Member selection is based on R = 0.80 kips.

For member AC (see Table XIV): C3x1.16
For member BC (see Tabls XIV): C3x1.16

(f) Determine connection details (see Figure 13A):

(1) Structural Steel

Beam splicing (see Section 4.7.1.2): Beam splicing, if required, shall be as per Figure 17.

Frame connections at C: Details are given in Figure 20 for connecting the top and bottom beams to members AC and for connecting members BC to members AC. Use welded connection for connecting members BC to members AC. Use 5/8 in. diam. ASTM A307 bolts and nuts for connecting the top and bottom beams to the clip angles attached to members AC (four bolts required for each beam; one each end and two at the center).

(2) Aluminum

Beam splicing (see Section 4.7.1.3): Beam splicing, if required, is the same as for structural steel, except aluminum shall be soldered where steel is welded.

Frame connections at C: Same as above for structural steel, except aluminum shall be soldered where steel is welded, and 1/2 in. diam. aluminum alloy 2024-T4 bolts and 6061-T6 nuts shall be used in place of the 5/8 in. diam. A307 bolts and nuts for connecting the top and bottom beams to the clip angles attached to members AC.

(3) Cold-formed Steel
Beam splicing (see Section 4.7.1.4 and Note at bottom of Table XII): Beam splicing, if required, is the same as for structural steel.

Frame connections at C: Details are given in Figure 35 for connecting the top and bottom beams to members AC. Use four-bolt connections with 4 in. by 5-1/2 in. by 1/4 in. thick ASTM A36 steel plates welded to the beams at the connection points. Use four 1/4 in. diam. ASTM A36 bolts and nuts at each connection. Details are given in Figure 32 for connecting members BC to members AC. Use welded connection.

(g) Determine bracing details:

(1) Structural Steel
Bracing shall be provided as indicated in Figure 23B for a two-bay, one-tier structure. Details for the bracing connections are given in Figures 24A and 24B. Use 3/8 in. diam. ASTM A36 rods threaded on both ends with 3/8 in. N.C. threads as the bracing members and ASTM A307 nuts and lockwashers. Connections to members BC shall be made in accordance with Figures 24A and 24B.

(2) Aluminum
Same as above for structural steel, except use 3/8 in. diam. aluminum alloy 6061-T6 rods threaded on both ends with 3/8 in. N.C. threads as the bracing members and 6061-T6 nuts and lockwashers. Also aluminum shall be soldered where steel is welded.

(3) Cold-formed steel
Bracing shall be provided as indicated in Figure 23B for a two-bay, one-tier structure. Details for the bracing connections are given in Figures 24A and 25. Use 3/8 in. diam. ASTM A36 rods threaded on both ends with 3/8 in. N.C. threads as the bracing members and ASTM A307 nuts and lockwashers. Use a 3-1/2 in. by 1-3/4 in. by 1/8 in. thick ASTM A36 steel baseplate welded to the center member BC for the mid-frame brace connections. Connections to members BC shall be made in accordance with Figures 24A and 25.

(h) Determine foundation required (See Section 4.7.1.5 and Table XXIII): Find the support point A and support point B rows in Table XXIII that correspond to a tilt angle of 50^{o}. The column with the smallest value of R that is greater than the R calculated for the top and bottom beam support points gives the foundation types required. In this case the foundation types required are:

At support point A: 30 c
At support point B: 26 c

Specifications for the foundation types are given in Table XXII and Figure 38.

Note: The foundation types required are the same for structural steel, aluminum and cold-formed steel.

(1) Foundation Details
For foundations at point A: Use type 30 c reinforced concrete pier with the following dimensions: diameter, 12 in.; height above finished grade, 12 in.; and depth below finished grade, 6.5 ft.

For foundations at point B: Use type 26 c reinforced concrete pier with the following dimensions: diameter, 18 in.; height above finished grade, 12 in.; and depth below finished grade, 5.33 ft.

Use number 5 reinforcing bar, arranged and tied as indicated in Figure 38.

(i) Determine connection details for members AC and BC to the foundation:

(1) Structural Steel
Foundation connections at point A (See Figures 39 and 40 and Table VI): For end frames and center frame: Use Type 1 connection with 4-1/2 in. by 4 in. by 1/4 in. thick ASTM A36 baseplate and 1/2 in. diam. by 6 in. long (N.C. thread) ASTM A307 anchor bolts and nuts. Embed bolts 4 in. into concrete.

Foundation connections at point B (See Figures 39 and 40 and Table VII): For end frames and center frame: Same as above, except use 5/16 in. thick steel plate.

(2) Aluminum
Foundation connections at points A and B are the same as for structural steel, except use aluminum alloy 6061-T6 baseplates soldered to members AC and BC. Bolts and embedment are the same as above.

(3) Cold-formed Steel
Foundation connections at points A and B (See Figures 39 and 47 and Tables XVI, XVIII and XIX): For end frames and center frame: Use Type 1 connection as per Figure 47 with 3-1/2 in. by 2-1/4 in. by 1/4 in. thick ASTM A36 steel baseplates and 1/2 in. diam. by 6 in. long (N.C. thread) anchor bolts and nuts. Embed bolts 4 in. into concrete.

(j) Basic cost analysis:
Determine amount of materials needed.
Lengths of members:

Beams = 42 ft.	Members AC = 2.3 ft
Members BC = 1.94 ft.	Bracing rods = 21.1 ft

(1) Structural Steel
ASTM A36 materials needed:

- 2 - 42 ft pieces of C3x4.1 sections (beams). Note: May need to use 21 ft pieces spliced together.
- 3 - 2.3 ft pieces of L1x1x1/8 (members AC) + 3 - 1.94 ft pieces of L1x1x1/8 (members BC).

- 2 - 3/8 in. diam. by 21.2 ft long rods, thread on both ends with N.C. threads (diagonal braces).
- 4 - 1-1/4 in. long pieces of L3x3x3/16 (clips for end frame members AC) + 2 - 6 in. long pieces of L3x3x3/16 (clip for center frame member AC).
- 4 - 1-1/2 in. long pieces of L1-1/2x1-1/2x1/8 (bracing connection brackets).
- 1 - 3-1/2 in. by 1-3/4 in. by 1/8 in. plate (center bracing support baseplate).
- 3 - 4 in. by 4-1/2 in. by 1/4 in. plates (baseplates for foundation connections at support A).
- 3 - 4 in. by 4-1/2 in. by 5/16 in. plates (baseplates for foundation connections at support B).

<u>ASTM A307 materials needed</u>:

- 8 - 5/8 in. diam. by 1-1/4 in. long (N.C.) bolts and nuts (for connecting beams to members AC).
- 8 - 3/8 in. N.C. nuts and lockwashers (for connecting brace rods to members BC).

<u>Other materials needed</u>:
<u>Welding rod sufficient</u> to weld the following:

- Clips to members AC (approximately 3 ft weld length).
- Members AC to members BC (approximately 1/2 ft weld length).
- Bracing support plate to center frame member BC (approximately 1-1/4 ft weld length).
- Foundation baseplates to members AC and members BC (approximately 2 ft weld length).

Total weld length = 6-3/4 ft

<u>Cost of materials</u>:

- 2 x 42 ft x 4.1 lb/ft = 344.4 lb
- (3 x 2.3 ft + 3 x 1.94 ft) 0.8 lb/ft = 10.18 lb
- $\frac{2 \times \pi}{4}$ (0.375 in.)^2 x $\frac{1 \text{ ft}^2}{144 \text{ in.}}$ x 21.1 ft x 490 lb/ft^3 = 15.9 lb
- (4 x 0.1 ft + 2 x 0.5 ft) 3.71 lb/ft = 5.2 lb

- 4×0.1 ft $\times$ 1.23 lb/ft = 0.51 lb

- $(3\text{-}1/2 \text{ in.} \times 1\text{-}3/4 \text{ in.} \times 1/8 \text{ in.}) \dfrac{1 \text{ ft}^3}{1728 \text{ in}^3} \times 490 \text{ lb/ft}^3 = 0.22 \text{ lb}$

- $3\,(4 \text{ in.} \times 4\text{-}1/2 \text{ in.} \times 1/4 \text{ in.}) \dfrac{1 \text{ ft}^3}{1728 \text{ in}^3} \times 490 \text{ lb/ft}^3 = 3.83 \text{ lb}$

- $3\,(4 \text{ in.} \times 4\text{-}1/2 \text{ in.} \times 5/16 \text{ in.}) \dfrac{1 \text{ ft}^3}{1728 \text{ in}^3} \times 490 \text{ lb/ft}^3 = 4.79 \text{ lb}$

- Welding rod and bolts, nuts and washers = 15 lb (approx.)

Total = 400 lb

Assume material and installation cost of $1000/ton, including welding rods and bolts, nuts and washers.

Material and installation (excluding welding): 400 lb x $1000/ton = $400.00

Welding: 6.75 ft (1/8 in. and 3/16 in. welds) x $0.06/L.F. = $4.05

Total bare cost estimate for structural steel (excluding foundation) = $404.05

(2) Aluminum

Aluminum alloy 6061-T6 materials needed:

- 2 - 42 ft pieces of C3x1.73 sections (beams). Note: May need to use 21 ft pieces spliced together.

- 3 - 2.3 ft pieces of L1-1/2x1-1/2x1/8 (members AC) + 3 - 1.94 ft pieces of L1-1/2x1-1/2x1/8 (members BC).

- 2 - 3/8 in. diam. by 21.1 ft long rods, thread on both ends with N.C. threads (diagonal braces).

- 4 - 1-3/4 in. long pieces of L3x3x3/16 (clips for end frame members AC) + 2 - 6 in. long pieces of L3x3x3/16 (clip for center frame member AC).

- 4 - 1-1/2 in. long pieces of L1-1/2x1-1/2x1/8 (bracing connection brackets).

- 1 - 3-1/2 in. x 1-3/4 in. by 3/16 in. plate (center bracing support baseplate).

- 3 - 4 in. by 4-1/2 in. by 1/4 in. plates (baseplates for foundation connections at support A).

- 3 - 4 in. by 4-1/2 in. by 5/16 in. plates (baseplates for foundation connections at support B).

Aluminum alloy 2024-T4 materials needed:

- 8 - 1/2 in. diam. by 1-1/4 in. long (N.C.) bolts and nuts (for connecting beams to members AC).
- 8 - 3/8 in. N.C. nuts and lockwashers (for connecting brace rods to members BC).

Soldering materials sufficient to solder the following:

- Clips to members AC (approximately 4 ft soldering length).
- Members AC to members BC (approximately 1 ft soldering length).
- Bracing support plate to center frame member BC (approximately 1 ft soldering length).
- Foundation baseplates to members AC and BC (approximately 3-1/2 ft soldering length).

Total soldering length = 9-1/2 ft

Cost of materials:

- $2 \times 42 \text{ ft} \times 1.73 \text{ lb/ft} = 145.3 \text{ lb}$
- $(3 \times 2.3 \text{ ft} + 3 \times 1.94 \text{ ft})\ 0.42 \text{ lb/ft} = 5.34 \text{ lb}$
- $\frac{2 \times \pi}{4} (0.375 \text{ in.})^2 \times \frac{1 \text{ ft}^2}{144 \text{ in.}^2} \times 21.1 \text{ ft} \times 165 \text{ lb/ft}^3 = 5.37 \text{ lb}$
- $(4 \times 0.15 \text{ ft} + 2 \times 0.6 \text{ ft})\ 1.28 \text{ lb/ft} = 2.3 \text{ lb}$
- $4 \times 0.15 \text{ ft} \times 0.42 \text{ lb/ft} = 0.25 \text{ lb}$
- $(3\text{-}1/2 \text{ in.} \times 1\text{-}3/4 \text{ in.} \times 3/16 \text{ in.}) \frac{1 \text{ ft}^3}{1728 \text{ in.}^3} \times 165 \text{ lb/ft}^3 = 0.11 \text{ lb}$
- $3\ (4 \text{ in.} \times 4\text{-}1/2 \text{ in.} \times 1/4 \text{ in.}) \frac{1 \text{ ft}^3}{1728 \text{ in.}^3} \times 165 \text{ lb/ft}^3 = 1.29 \text{ lb}$
- $3\ (4 \text{ in.} \times 4\text{-}1/2 \text{ in.} \times 5/16 \text{ in.}) \frac{1 \text{ ft}^3}{1728 \text{ in.}^3} \times 165 \text{ lb/ft}^3 = 1.61 \text{ lb}$

- Soldering rod and bolts, nuts and washers = 12 lb (approx.)

Total = 174 lb

Assume material and installation cost of \$2.30/lb including solder and nuts, bolts and washers.

Material and installation (excluding soldering): 174 lb x \$2.30/lb = \$400.20

Soldering: 9.5 ft x \$0.60/L.F. = \$5.70

Total bare cost estimate for aluminum (excluding foundation) = \$405.90

(3) Cold-formed Steel
ASTM A245C, A446A, A570C and/or A611C materials needed:

- 2 - 42 ft pieces of C5x2.53 sections (beams). Note: May need to use 21 ft pieces spliced together.

- 3 - 2.3 ft pieces of C3x1.16 (members AC) + 3 - 1.94 ft pieces of C3x1.16 (members BC).

- 2 - 3/8 in.diam. by 21.1 ft long rods, thread on both ends with N.C. threads (diagonal braces).

ASTM A36 materials needed:

- 6 - 4 in. by 5-1/2 in. by 1/4 in. plates (to connect beams to members AC)

- 4 - 1-1/2 in. long pieces of L1-1/2x1-1/2x1/8 (bracing connection brackets)

- 1 - 3-1/2 in. by 1-3/4 in. by 1/8 in. plate (center bracing support baseplate)

- 6 - 3-1/2 in. by 2-1/4 in. by 1/4 in. plates (baseplates for foundation connection at supports A and B)

ASTM A307 materials needed:

- 24 - 1/4 in. diam. by 1-1/4 in. long (N.C.) bolts and nuts (for connecting beams to members AC).

- 8 - 3/8 in. N.C. nuts and lockwashers (for connecting brace rods to members BC).

Other material needed:
Welding rod sufficient to weld the following:

- Plates to beams (approximately 4 ft weld length).

- Members AC to members BC (approximately 3 ft weld length).

- Bracing support plate to center frame member BC (approximately 1 ft weld length).
- Foundation baseplates to members AC and members BC (approximately 4 ft weld length).

Total weld length = 12 ft

Cost of materials:

- 2 x 42 ft x 2.53 lb/ft = 212.52 lb
- (3 x 2.3 ft + 3 x 1.94 ft) 1.16 lb/ft = 14.76 lb
- $\frac{2 \times \pi}{4}$ (0.375 in.)^2 x $\frac{1\ \text{ft}^2}{144\ \text{in.}^2}$ x 21.1 ft x 490 lb/ft^3 = 15.86 lb
- 6 (4 in. x 5 $\frac{1}{2}$ in. x $\frac{1}{4}$ in.) $\frac{1\ \text{ft}^3}{1728\ \text{in.}^3}$ x 490 lb/ft^3 = 9.36 lb
- 4 x 0.1 ft x 1.23 lb/ft = 0.49 lb
- (3 $\frac{1}{2}$ in. x 1 $\frac{3}{4}$ in. x $\frac{1}{8}$ in.) $\frac{1\ \text{ft}^3}{1728\ \text{in.}^3}$ x 490 lb/ft^3 = 0.22 lb
- 6 (3 $\frac{1}{2}$ in. x 2 $\frac{1}{4}$ in. x $\frac{1}{4}$ in.) $\frac{1\ \text{ft}^3}{1728\ \text{in.}^3}$ x 490 lb/ft^3 = 3.35 lb
- Welding rod and bolts, nuts and washers = 16 lb (approx.)

Total = 273 lb

Assume material and installation cost of $1100/ton, including welding rods and bolts, nuts and washers.

Materials and installation (excluding welding): 273 lb x $1100/ton = $150.15

Welding: 12 ft (1/8 in. and 3/16 in, welds) x $0.60/L.F. = $7.20

Total bare cost estimate for cold-formed steel (excluding foundation) = $157.35

(4) Foundations:
Reinforced concrete needed:

- Type 30 c: $3 \times \frac{\pi}{4} (1\ ft)^2 \times 7.5\ ft = 17.67\ ft^3$
- Type 26 c: $3 \times \frac{\pi}{4} (1.5\ ft)^2 \times 6.33\ ft = 33.56\ ft^3$

Total = 51.23 ft^3 = 1.90 cu yd

Anchor bolts (including material and installation):

6 (1/2 in. diam. by 6 in. long (N.C.) bolts and nuts) x \$3.48 ea. = \$20.88

Assume cost of \$135/cu yd for concrete, rebar and installation (including forming, but excluding anchor bolts)

1.9 cu yd x \$135/cu yd = \$256.50

Anchor bolts = \$20.88

Total bare cost estimate for foundations = \$277.38

(5) Summary:

Structural Steel System Cost:

Structure Cost = \$404.05
Foundation Cost = \$277.38
Total Bare Cost Estimate = \$681.43
25 percent for O+P = \$170.36
15 percent for Contingency = \$102.21
Total Estimated Cost = \$954.00

Aluminum System Cost:

Structure Cost = \$405.90
Foundation Cost = \$277.38
Total Bare Cost Estimate = \$683.28
25 percent for O+P = \$170.82
15 percent for Contingency = \$102.49
Total Estimated Cost = \$956.59

Cold-formed Steel System Cost:

Structure Cost = \$157.35
Foundation Cost = \$277.38
Total Bare Cost Estimate = \$434.73
25 percent for O+P = \$108.68
15 percent for Contingency = \$65.21
Total Estimated Cost = \$608.62

Based on this analysis, the cold-formed structural steel system should be chosen. Other considerations, however, may dictate use of one of the other systems.

Note: The above cost analysis is based on estimated costs in 1979 dollars and includes only the bare costs for the structures and their foundations.

5.2 Design Example 2: 200 Wp Dedicated (Metric Calculations)

A. Description

Site location:	Near Lima, Peru (Latitude = 12° S, Longitude = 75.3° W)
Site conditions:	Level site, no size or shape restrictions on array field, and no shading from nearby tress or structures.
Weather conditions:	Temperate, semi-arid area with temperature range of approximately 5° to $25^{\circ}C$ (no frost or snow); assume maximum wind velocity of 130 km/hr.
Tilt angle:	Design for fixed tilt of 12° (see Step 1, item (b) in Section 2.0).
Array orientation:	Orient array so that it faces due North.

B. Module Determination

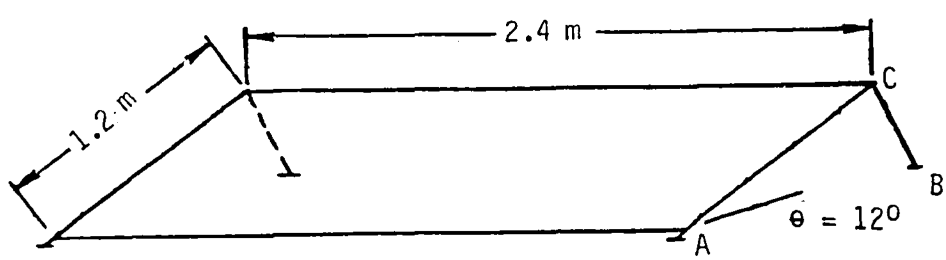
34.5 watts (peak) module (7.2 percent efficiency)
Size: 0.4 m x 1.2 m
Weight: 8.2 kg/m^2

C. Array Size

Six 34.5-watt modules are required for the array. This assumes that this combination of modules will provide the desired voltage (see Step 1, item (b) in Section 2.0). In this case the array size is 2.88 m^2.

D. Array Layout and Structural Material Selection

Since there are no apparent restrictions for the site, a one-tier one-bay structure was chosen for this example. The structural system chosen is illustrated in the following figure:



Use structural steel as the structural material for this example.

E. Loads

(a) Dead Load:

DL = weight of structure + weight of modules

$$DL = 200\ N/m^2\ (est.) + 8.2\ \frac{kg}{m^2} \times \frac{9.8\ N}{kg} = 280.36\ N/m^2$$

(b) Live and other loads: The maximum wind velocity for the site is 130 km/hr. Using Table 3.7 of the reference given in Section 7.4 and converting to English Units and then back to metric units yields a wind pressure of 785 N/m^2 at heights of less than 9 meters. The seismic load is $V = 0.24\ DL = 0.24 \times 280.36\ N/m^2 = 67.3\ N/m^2$. There are no snow or ice loads for the site. The maintenance load is 718 N/m^2 (See Section 4.3.2.).

Using the method given in Section 4.3.9:

Total Load = Dead Load + Wind Load + 280.36 N/m^2 + 785 N/m^2 = 1065 N/m^2.

Use total load of 1070 N/m^2 as the design criterion.

F. Structural Design

See Section 4.7.1.2, Structural Design Procedure and Figures 11A, 14B, and 16A

(a) Determine W: Uniform load applied to top and bottom beams.

$$W = \text{Total Load} \times L'/2$$

where

L' height of one module

$$W = 1070\ N/m^2 \times 1.2\ m \times 1/2 = 642\ N/m$$

(b) Determine M: The maximum moment developed in top and bottom beams. For single span beams:

$$M = 1.25\ WL^2/8$$

where

L span length between support frames

$$M = 1.25 \times 642\ N/m \times (2.4\ m)^2 \times 1/8 = 577.8\ N\text{-}m$$

(c) Determine R: The maximum reaction for the top and bottom beams. For single span beams:

$$R = WL/2$$

where

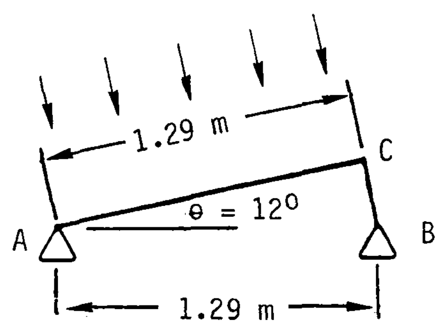
L span length between support frames

$R = 642 \text{ N/m} \times 2.4 \text{ m} \times 1/2 = 770.4 \text{ N}$

(d) Determine sections to be used for top and bottom beams.

Use Structural Steel
For top and bottom beams (See Table III): Use L2-1/2x2x3/16 or metric equivalent.

(e) Determine lengths of members AC and BC.



AC = AB = height of module + 7.6 cm + 1.3 cm = 1.29 m = L"; tilt = 12^{o} up from horizontal.

$$BC = \sqrt{2\ L''^{2} - 2\ L''\cos\theta}$$

$$BC = \sqrt{2\ (1.29)^{2}\ (1.29)^{2} \cos 12^{o}} = 0.270 \text{ m}$$

(f) Determine sections to be used for members AC and BC.

Use Structural Steel
For Member AC (See Table IV): Use L1x1x1/8 or metric equivalent.
For Member BC (See Table IV): Use L1x1x1/8 or metric equivalent.

(g) Determine connection details (See Figure 13A):

Frame connections at C: Details are given in Figure 19 for connecting the top and bottom beams to members AC and for connecting members BC to members AC. Use bolted connection (10 mm diam. ASTM A325 bolts and nuts, or equivalent) for connecting members BC to members AC. Use 12 mm diam. ASTM A307 bolts and nuts for connecting the top and bottom beams to the clip angles attached to members AC.

Note: Since this is a relatively small structure at a very low tilt angle, diagonal bracing in the "BC" plane is not needed.

(h) Determine foundation required (See Section 4.7.1.5 and Table XXIII): Foundation types required are:

At support point A: 39 c
At support point B: 29 c

Specifications for the foundation types are given in Table XXII and Figure 38.

Foundation Details
For foundations at point A: Use type 39 c reinforced concrete pier with following dimensions: diameter, 30 cm; height above finished grade, 0.15 m; and depth below grade, 0.84 m.

For foundations at point B: Use type 29 c reinforced concrete pier with following dimensions: diameter, 30 cm; height above finished grade, 0.15 m; and depth below grade, 0.91 m.

Use number 5 reinforcing bar, arranged and tied as indicated in Figure 38.

(i) Determine connection details for members AC and BC to the foundation:

Foundation connection at point A (See Figures 39 and 40 and Table VI). Use Type 1 connection with 11.4 cm by 10 cm by 6.53 mm thick ASTM A36 baseplate and 12.7 mm diam. x 15 cm long ASTM A307 anchor bolts and nuts. Embed bolts 10.2 cm into concrete.

Foundation connection at point B (See Figures 39 and 40 and Table VII): Same as above, except use 7.94 mm thick steel plate.

5.3 Design Example 3: 4 kWp Dedicated (English Calculations)

A. Description

Site location:	Tucson, Arizona (Latitude = 32.2^o N, Longitude = 111^o W).
Site conditions:	Level site, no size or shape restrictions on array field, and no shading from nearby trees or structures.
Weather conditions:	Consult tables and figures.
Tilt angle:	Design for permanent (fixed) tilt of 30^o (see Step 1, item (b) in Section 2.0).
Array orientation:	Orient array so that it faces due south.
Other information:	Space structures so as to avoid array shading after 0900 hours local time.

B. Module Determination

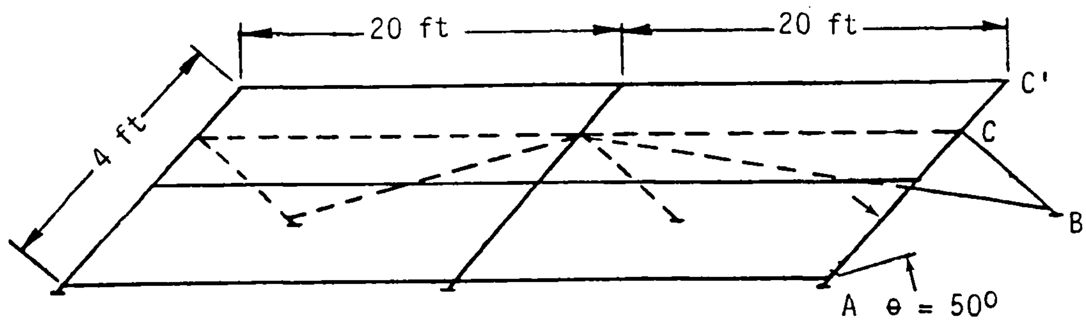
25 watt (peak) module (6.7 percent efficiency)
Size: 2 ft x 2 ft
Weight: 2.5 lb/ft^2

C. Array Size

160 25-watt modules are required for the array. Therefore, the array size must be 160 x 2 ft x 2 ft = 640 ft^2. This assumes that this combination of modules will provide integral sets of modules (strings) to produce the desired voltage. For this example, assume a string consists of eight modules (See Step 1, item (b) in Section 2.0).

D. Array Layout and Structural Material Selection

Since there are no apparent restrictions for the site, a two-tier two-bay structural system consisting of four rows was chosen for this example. This arrangement requires four structures 40 ft long with each structure housing 40 solar cell modules. The structural system chosen is illustrated in the following figure:



Use structural steel as the structural material.

E. Loads

(a) Dead Load:

DL = Weight of structure + Weight of modules

DL = 4 psf (est.) + 2.5 psf = 6.5 psf

(b) Live and other loads: The wind map (Figure 9) indicates a 20 psf wind pressure for the site. Design for 15 psf since structure is less than 30 ft above ground. The seismic load, V = 0.24 DL = 0.24 x 6.5 psf = 1.56 psf. The snow map (Figure 10) indicates a 5 psf snow pressure for the site. Reduction is not done because snow load is less than 15 psf. Ice loads are not considered significant for the area. Maintenance load is 15 psf.

Using the method given in Section 4.3.9:

Total Load = Dead Load + Wind Load = 6.5 psf + 15 psf = 21.5 psf

Use total load of 22 psf as the design criterion.

F. Structural Design

See Section 4.7.1.1, Structural Design Procedure and Figures 11D, 14A, and 16B

(a) Determine W: Uniform load applied to the beams.
For top and bottom beams:

W = Total Load x $L'/2$

where

L' height of one module

W = 22 psf x 2 ft/2 = 22 lb/ft

For middle beam:

W = Total Load x L'

W = 22 psf x 2 ft = 44 lb/ft

(b) Determine M: The maximum moment developed in the beams.
For two continuous span beams
For top and bottom beams:

$M = 1.25\ LW^2/8$ = 1.25 x 22 lb/ft x $(20\text{ ft})^2$ x 1/8 x 12 in./ft x 0.001 k/lb = 16.5 k-in.

For middle beam:

$M_m = M_{TB} \times 2$ = 16.5 k-in. x 2 = 22 k-in.

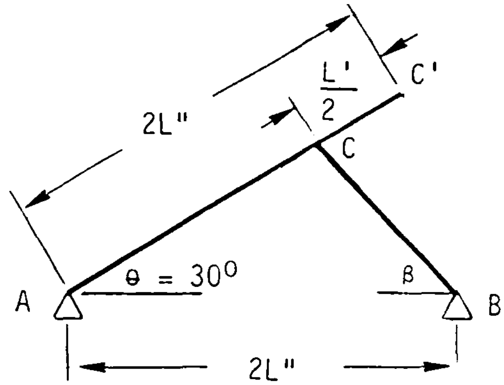
(c) Determine R: The maximum reaction for the top and bottom beams.
For two continuous span beams:

$R = 1.25\ WL$ = 1.25 x 22 lb/ft x 20 ft x 0.001 k/lb = 0.550 k = 1.25 WL

(d) Determine sections to be used for top, bottom, and middle beams.

Use Structural Steel
Top and bottom beams (See Table III): Use C3x4.1.
Middle beam (See Table III): Select a T (tee) or wide flange; depth of top and bottom beams can be adjusted when connected to members AC' so that top surfaces align with top surface of middle beam when beams chosen are of different depths. Use WT5x6.

(e) Determine lengths of members AC' and BC.



AC' = AB = height of two modules + 3 in. = 4.25 ft = 2 L", tilt = 30° up from horizontal.

$$BC = \sqrt{6.25\ L''^2 - 6\ L''^2 \cos\theta}$$

$$BC = \sqrt{6.25\ (4.25/2\ \text{ft})^2 - 6\ (4.25/2\ \text{ft})^2 \cos 30^o} = 2.18\ \text{ft}$$

(f) Determine sections to be used for members AC' and BC.
For member AC' (See Table V): Use C3x4.1.
For member BC (See Table V): Use L1-1/2x1-1/2x1/8.

(g) Determine connection details (See Figure 16B)

Beam splicing (See Section 4.7.1.2): Beam splicing, if required, shall be as per Figure 17.

Beam connections to members AC': Connection of the top and bottom beam to members AC' are identical to that for one-tier structures. The details for these connections may be developed from Figure 20. Details for connecting the middle beam to members AC' are given in Figure 21. Use 5/8 in. diam ASTM A307 bolts and nuts for connecting the beams to the clip angles attached to members AC'.

Frame connections at C: Details for connecting members BC to members AC are given in Figure 22A. Use welded connection, as indicated.

(h) Determine bracing details:
Bracing shall be provided as indicated in Figure 23B for a two-bay, two-tier structure. Details for the bracing requirements may be developed from Table VIII and Figures 24A, 25, 29, and 30. Use 3/8 in. diam. ASTM A36 rods threaded on both ends with 3/8 in. N.C. threads as both the horizontal strut and diagonal bracing members, and ASTM A307 nuts and lockwashers. Use 3/16 in. thick ASTM A36 steel baseplates welded to the center frame members (AC' and BC) for the mid-frame horizontal strut and diagonal bracing connections. Connections to members AC' and BC shall be made in accordance with Figures 24A, 25, 29, and 30.

(i) Determine foundation required (See Section 4.7.1.5 and Table XXIV): Foundation types required are:

At support point A: 18 c
At support point B: 41 c

Specifications for the foundation types are given in Table XXII and Figure 38.

Foundation Details

For foundations at point A: Use type 18 c reinforced concrete pier with following dimensions: diameter, 18 in.; height above finished grade, 12 in.; and depth below grade, 5.75 ft.

For foundations at point B: Use type 41 c reinforced concrete pier with following dimensions: diameter, 24 in.; height above finished grade, 12 in.; and depth below grade, 5.50 ft.

Use number 5 reinforcing bar for type 18 c and number 7 for type 41 c, arranged and tied as indicated in Figure 38.

(j) Determine connection details for members AC' and BC to the foundation:

Foundation connections at points A and B may be developed from Figures 39 and 41 and Tables VI and VII. Use 1/4 in. thick ASTM A36 baseplate and 1/2 in. diam. by 6 in. long (N.C. thread) ASTM A307 anchor bolts and nuts. Embed bolts four inches into concrete.

(k) Determine row spacing to avoid shading after 0900 hours on December 22. Using the method given in section 3.5.1:
From Tables IIA and IIB:
$d = (-23.45^\circ)$, ET = 0.030 hours and TZN = +7

$$h = 15\ (t - 12 + TZN + ET) - Long$$

$$h = 15^\circ/hr\ (9 - 12 + 7 + 0.03)\ hr - 111^\circ = 50.55^\circ$$

$$h' = \cos^{-1}\ (-\tan Lat.)(\tan d)$$

$$h' = \cos^{-1}\ [-\tan 32.2^\circ][\tan (-23.45^\circ)] = 74.15^\circ$$

$|h| < |h'|$, therefore the time chosen to avoid shading is after sunrise.

Sunrise at the site on December 22 is:

$$SRT = 12 - (h'/15) - ET - TZN + (Long/15)$$

$$SRT = 12 - 74.15/15 - 0.030 - 7 + (111/15) = 7.427\ hr = 7{:}26\ a.m.\ MST$$

$$SALT = \sin^{-1}\ (\cos Lat.\ \cos h\ \cos d + \sin Lat.\ \sin d)$$

$$SALT = \sin^{-1} [\cos 32.2^o \cos (-50.55^o) \cos (-23.45^o) + \sin 32.2^o \sin (-23.45^o)]$$

$$SALT = 16.33^o$$

$$SAZM = \sin^{-1} [\cos d \sin h/\cos (SALT)]$$

$$SAZM = \sin^{-1} [\cos (-23.45^o) \sin (-50.55^o)/\cos 16.33^o]$$

$SAZM = -47.58^o$ or 47.58^o East of South.

$$SPACE_1 = \frac{Y \sin (\delta + \tau)}{\sin \gamma}$$

$$a = \left| SAZM + CAZM \right| = \left| -47.58^o + 0^o \right| = 47.58^o$$

$$\delta = \text{arc tan} \left[\frac{\tan (SALT)}{\cos a}\right] = \text{arc tan}\left[\frac{\tan 16.33^o}{\cos 47.58^o}\right] = 23.47^o$$

$$SPACE_1 = \frac{(4 \text{ ft}) \sin (23.47^o + 30^o)}{\sin 23.47^o} = 8.06 \text{ ft}$$

$$SPACE_2 = \frac{X \sin (\delta + \tau)}{\sin \delta} \times \frac{1}{\sin \tau \cot (SALT) \sin a}$$

$$SPACE_2 = \frac{(40 \text{ ft}) \sin (23.47^o + 30)^o}{\sin 23.47^o} \times \frac{1}{\sin 30^o \cot 16.33^o \sin 47.58^o}$$

$$SPACE_2 = 64.06 \text{ ft}$$

Since either $SPACE_1$ or $SPACE_2$ ensures no shading, use 8.06 ft as spacing between the rows in the South to North direction.

5.4 Design Example 4: 1 kWp Portable (Metric Calculations)

A. Description

Site location:	Tangaye, Upper Volta (Latitude = 12.5^o N, Longitude = 0^o)
Site conditions:	This is to be a portable array capable of being moved from site to site with a minimum of effort. It is assumed that all sites will be level with no size or shape restrictions on the array field and selected so as to avoid shading from nearby trees or structures.
Weather conditions:	Temperature range of 7^o to 49^o C (no frost or snow); maximum wind velocity = 130 km/hr.
Tilt angle:	Design for fixed tilt of 12.5^o (see Step 1, item (b) in Section 2.0).
Array orientation:	Assume array will be oriented so that it faces due south.

B. Module Determination

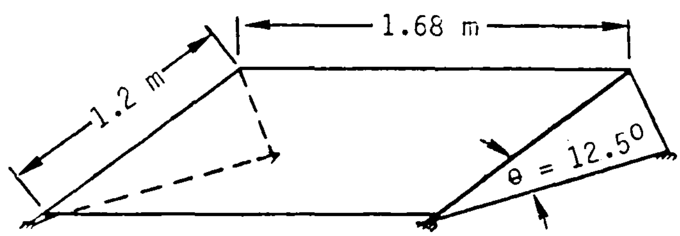
84 watt (peak) module (12.5 percent efficiency)
Size: 0.56 m x 1.2 m
Weight: 13.6 kg/m^2

C. Array Size

Twelve 84-watt modules are required for the array. This assumes that this combination of modules will provide the desired voltage (see Step 1, item (b) in Section 2.0). In this case the array size is 8 m^2.

D. Array Layout and Structural Material Selection

Since the array is intended to be portable, i.e., capable of being easily moved from one site to another within a locality, a one-tier, one-bay structural system consisting of four portable units was chosen for this example. This arrangement requires four separate structures 1.68 meters long with each structure housing three solar cell modules. The structural system chosen is illustrated in the following figure:



Use aluminum as the structural material.

E. Loads

(a) Dead Load:

DL = Weight of structure + Weight of modules

DL = 100 N/m^2 (est.) + 13.6 kg/m^2 x 9.8 N/kg = 233.28 N/m^2

(b) Live and other loads: The maximum wind velocity for the locality is 130 km/hr. Using Table 3.7 of the reference given in Section 7.4 yields a wind pressure of 785 N/m^2 at heights of less than 9 meters. The seismic load is V = 0.24 DL = 0.24 x 233.38 N/m^2 = 56 N/m^2. There are no snow or ice loads for the locality. The maintenance load is 718 N/m^2 (See Section 4.3.2).

Using the method given in Section 4.3.9:

Total Load = Dead Load + Wind Load = 233.28 N/m^2 + 785 N/m^2
= 1018.28 N/m^2

Use total load of 1020 N/m^2 as the design criterion.

F. Structural Design

See Section 4.7.1.1, Structural Design Procedure and Figures 13D and 16A

(a) Determine W: Uniform load applied to top and bottom beams.

$W = \text{Total Load} \times L'/2$

where

L' height of one module

$W = 1020\ N/m^2 \times 1.2\ m \times 1/2 = 612\ N/m$

(b) Determine M: The maximum moment developed in top and bottom beams. For single span beams:

$M = 1.25\ WL^2/8$

where

L span length between support frames

$M = 1.25 \times 612\ N/m \times (1.68\ m)^2 \times 1/8 = 269.9\ N\text{-}m$

(c) Determine R: The maximum reaction for the top and bottom beams. For single span beams:

$R = WL/2$

where

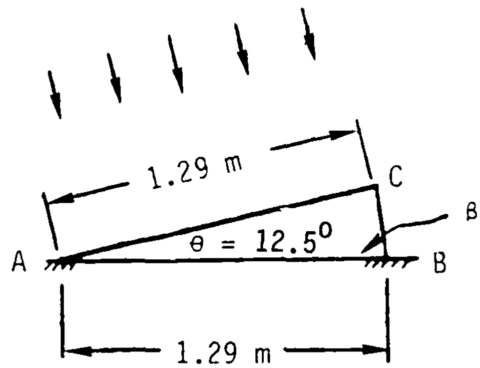
L span length between support frames

$R = 612\ N/m \times 1.68\ m \times 1/2 = 514\ N$

(d) Determine sections to be used for top and bottom beams:
Use Aluminum
Top and bottom beams (see Table IX): Use L2x1-1/2x1/8 or metric equivalent.

(e) Determine lengths of members AC and BC.

(e) Determine lengths of members AC and BC.



AC = AB = height of module + 7.6 cm + 1.27 cm = 1.29 m = L"; tilt = 12.5° up from horizontal.

$$BC = \sqrt{2\,L''^2 - 2\,L''^2 \cos\theta}$$

$$BC = \sqrt{2\,(1.29)^2 - 2\,(1.29)^2 \cos 12.5^\circ} \quad = 0.281 \text{ m}$$

(f) Determine sections to be used for members AC and BC.
<u>Use Aluminum</u>
<u>For Member AC</u> (see Table X): Use L1-1/2x1-1/2x1/8 or metric equivalent.
For Member BC (see Table X): Use L1-1/2x1-1/2x1/8 or metric equivalent.

(g) Determine anchoring requirements for the portable structures:

<u>For one-tier structure</u>
<u>Uplift and lateral loads at support points A and B due to wind:</u>

$B'_x = R'/\tan\beta$ (See Section 4.7.1.1)

$B'_y = R'$

$A'_x = 2\,R'\,\sin\theta - R'/\tan\beta$

$A'_y = 2\,R'\,\cos\theta - R'$

$\beta = (180 - \theta)/2 = (180^\circ - 12.5^\circ)/2 = 83.75^\circ$

<u>Determine wind load W'</u>: uniform wind load applied to top and bottom beams.

W' = wind load x L'/2

where

L height of one module

W' = 785 N/m^2 x 1.2 m x 1/2 = 471 N/m

Determine R': The maximum reaction for the top and bottom beams due to wind load alone.

For single span beams:

$R' = W'L/2$

where

L span length between support frames

$R' = 471 \text{ N/m} \times 1.68 \text{ m} \times 1/2 = 395.64 \text{ N}$

Lateral loads at B due to wind:

$B'_x = (395.64 \text{ N})/\tan 83.75^o = 43.33 \text{ N}$

Uplift loads at B due to wind:

$B'_y = R' = 395.64 \text{ N}$

Lateral loads at A due to wind:

$A'_x = 2\ (395.64 \text{ N}) \sin 12.5^o - 395.64 \text{ N}/\tan 83.75$

$A'_x = 127.93 \text{ N}$

Uplift loads at A due to wind:

$A'_y = 2\ (395.64 \text{ N}) \cos 12.5^o - 395.64 \text{ N}$

$A'_y = 376.88 \text{ N}$

Therefore, the weight which must be placed at support B to resist both uplift and sliding due to wind is 396 N. The weight which must be placed at support A to resist both uplift and sliding due to wind is 377 N.

One option is to connect the frame support points A and B together with spreader bars and then apply weights, such as sand bags, to the bars to provide resistance to overturning and/or sliding due to wind. In this case, it is assumed that the sand bags are concentrated as close as possible to the support points. If the weights are concentrated within 1/3 meter of the support points A and B, then the maximum moment in the spreader-bars is

$M_{\substack{max \\ BAR}} = 1/3 \text{ m} \times 396 \text{ N} = 132 \text{ N-m}$

From Table IX: Use L1-3/4x1-1/4x1/8 or metric equivalent.

Note: If the structures are to be left in one place for a year or more, it is suggested that the weights be increased by 1-1/2 times the calculated weights needed to resist uplift and sliding. Also if the structures are to be relocated to sites outside the locality for which they were designed, then they should be reevaluated for the wind conditions for the new site.

(h) Determine connection details (see Figures 13D and 16A)

Frame connections at C: Details are given in Figure 19 and Section 4.7.1.3 for connecting the top and bottom beams to members AC and for connecting members BC to members AC. Use bolted connection (1.27 cm diam. aluminum alloy 2024-T4 bolts and 6061-T6 nuts, or equivalent) for connecting members BC to members AC and for connecting the spreader-bars to members AC and BC. Use 1.27 cm diam. aluminum alloy 2024-T4 bolts and 6061-T6 nuts, or equivalent, for connecting the top and bottom beams to the clip angles attached to members AC.

Note: Since this is a relatively small structure at a very low tilt angle, diagonal bracing in the BC plane is not needed.

5.5 Design Example 5: 200 Wp Pole-Mounted (English Calculations)

A. Description (See Section 4.7.2)

Site location:	Hawaii (Latitude = 18°54'N, Longitude = 155 5'W)
Site conditions:	Level site, clear of obstructions
Weather conditions:	Average temperature 65 to 70° F (no frost or snow); maximum wind velocity = 80 mph.
Tilt angle:	Maximum variable tilt that will be used is 34 .
Array orientation:	Orient array so that it faces due south.

B. Module Determination

20 watt (peak) module (7.5 percent efficiency)
Size: 3/4 ft x 4 ft
Weight: 2.8 lb/ft^2

C. Array Size

Ten 20-watt modules are required for the array. This assumes that this combination of modules will provide the desired voltage (see Step 1, item (b) in Section 2.0). In this case the array size is 30 ft^2.

D. Array Layout, Height of Pole and Structural Material Selection

Use single row of ten modules in side-by-side configuration., This means that the array must be 4 ft high (L') by 7.5 ft wide (L).

Use a pole height (HT) of 6 ft.

Use aluminum as the structural material.

E. Loads

(a) Dead Load:

DL = weight of structure + weight of modules

DL = 3 psf (est.) + 2.8 psf = 5.8 psf

(b) Live and other loads: The maximum wind velocity given for the site is 80 mph. From Table 3.7 of the reference given in Section 7.4, this corresponds to a wind load of 16.4 psf at heights of less than 30 ft. The seismic load is V = 0.24 DL = 0.24 x 5.8 psf = 1.4 psf. There are no snow or ice loads for the site. The maintenance load is 15 psf (see Section 4.3.2).

Using the method given in Section 4.3.9:

Total Load = Dead Load + Wind Load = 5.8 psf + 16.4 psf = 22.2 psf.

Use total load of 23 psf as design criterion.

F. Structural Design

See Section 4.7.2.1, Structural Design Procedure and Figures 12, 48 and 49

(a) Determine W: uniform load applied to the beams.
For top and bottom framing beams:

W = Total Load x L'/2

where

L' height of one module

W = 23 psf x 4 ft/2 = 46 lb/ft

(b) Determine F_R: The maximum resultant force at top of pole.

F_R = 1.25 P x L x L"

where

P total load
L span length
L" nominal distance between top and bottom beams

F_R = 1.25 x 23 lb/ft^2 x 7.5 ft x 4 ft = 863 lb

(c) Determine pole size (see Table XXVI):
Based on a maximum F_R of 863 lb, a maximum tilt angle of 34^o, and a pole height of 6 ft, the pole size required is:

Pole size: 3-1/2 #40 pipe.

(d) Determine sections to be used for framing members.

(1) Top and bottom framing beams (see Table XXXII):

$M = 1.25\ WL^2/8$
$= 1.25\ (46\ \text{lb/ft})\ (7.5\ \text{ft})^2 \times 0.001\ \text{k/lb} \times 12\ \text{in./ft} \times 1/8$
$= 4.86$ k-in.

Use: L2x2x1/4

(2) Side framing beam (see Table XXXIII):

$M = 1.25\ WLL'/4$
$= 1.25\ (46\ \text{lb/ft})\ (7.5\ \text{ft})\ (4\ \text{ft}) \times 0.001\ \text{k/lb} \times 12\ \text{in./ft} \times 1/4$
$= 5.18$ k-in.

Use: L2x2x1/4

(3) Center beam (see Table XXXIV):

$M = 1.25\ WL^2/2$
$= 1.25\ (46\ \text{lb/ft})\ (7.5\ \text{ft})^2 \times 0.001\ \text{k/lb} \times 12\ \text{in./ft} \times 1/2$
$= 19.40$ k-in.

Use TS2.25x2.25x0.156 with 1/4 in. thick trunion plates.

(4) Angle adjusting member (see Figure 49):
Use L1-1/2x1-1/2x1/8.

(e) Determine connection requirements.
Details for connecting the top and bottom beams to the side framing beams, the center beam to the side framing beams, center beam to top of pole, and connection of the angle adjustment member to the frame and pole are given in Figures 48 and 49.

Bonded connections shall be soldered.

Bolted connections shall consist of aluminum alloy 2024-T4 bolts and 6061-T6 nuts.

Bolt sizes shall be as follows:
Top/Bottom beam to side frame: Use 3/8 in. diam. bolts
Center beam trunion plate to side frame: Use 1/2 in. diam. bolts
Angle adjustment member to frame: Use 3/8 in. diam. bolts
Angle adjustment member to pole: Use 3/8 in. diam. bolts

(f) Determine foundation and baseplate connection requirements (see Figures 48, 50 and 51):

$M = 1.25\ F_R \sin\theta \times HT$

where

HT height of pole

$M = 1.25 \times 863 \text{ lb} \times \sin 34^{\circ} \times 6 \text{ ft} \times 0.001 \text{ k/lb} \times 12 \text{ in./ft} = 43.4$
$= 43.4$ k-in.

(1) Foundation details (Table XXXV):
Use reinforced concrete pier with the following dimensions: diameter, 2 ft; height above finished grade, 2 in.; and depth below finished grade, 4 ft. Use reinforcing bar arranged and tied as indicated in Figure B at the top of Table XXXV.

(2) Foundation connection details (Table XXXIX):
Use four-bolt connection with 7.6 in. by 7.6 in. by 3/4 in. thick aluminum alloy 6061-T6 baseplate soldered to the bottom of the aluminum mounting pole. Use 3/4 in. diam. by 7 in. long (N.C. thread) ASTM A307 anchor bolts and nuts. Embed bolts 4 inches into concrete.

6.0 GLOSSARY

6.1 Symbols and Acronyms

AISC	American Institute of Steel Construction
AITC	American Institute of Timber Construction
ASTM	American Society for Testing and Materials
AWS	American Welding Society
BBC	Basic Building Code (also known as BOCA)
cm	centimeter
cu yd	cubic yard
F_R	resultant force developed at top of pole under design load
FT (ft)	feet
HR (hr)	hours
HT	height of pole
I	current
IN (in.)	inches
JPL	Jet Propulsion Laboratory
K (k)	kip (1000 lbf) or kilo (1000)
kgf	kilograms force
km/hr	kilometer per hour
L	span length between support frames
L'	height of one module (Figure 14)
L"	nominal distance between beams (Figure 13 or Table IV)
L_c	maximum unbraced length of compression member
lbf	pounds force
M_m	maximum moment developed in middle beam under loads
M_{max}	maximum moment capacity
M_{TB}	maximum moment developed in top and bottom beams
MPH (mph)	miles per hour
N	Newton
NASA	National Aeronautics and Space Administration
NBC	National Building Code
NEC	National Electric Code
NOAA	National Oceanic and Atmospheric Administration
P	combined loads
PSF (psf)	lbf/ft^2
R	reaction load carrying capacity
R_{TB}	reaction developed in top and bottom beams under design load

SSBC	Southern Standard Building Code
T	Temperature (°C)
UBC	Uniform Building Code
V	Voltage (volts)
w	uniform load applied to beams
W_m	uniform load applied to middle beam
W_p	peak power (watts)
W_{TB}	uniform load applied to top and bottom beams
θ	tilt angle (degrees up from horizontal) = τ (Section 3.5.1 and Figure 8C)
ϕ	bolt size (diameter)

6.2 Conversions

A. Length

1 meter (m) = 100 centimeters (cm) = 1000 millimeters (mm) = 3.2808 ft
1 inch (in.) = 2.54 cm
1 foot (ft) = 0.3048 m

B. Area

1 ft^2 = 0.0929 m^2
1 m^2 = 10.764 ft^2

C. Volume

1 ft^3 = 0.0283 m^3
1 m^3 = 35.313 ft^3

D. Force

1 Newton (N) = 0.2248 lbf = 0.102 kgf
1 kgf = 9.807 N
1 lbf = 4.448 N
1 kip (k) = 1000 lbf = 4448 N

E. Pressure

1 N/m^2 = 0.02089 lbf/ft^2 (PSF)
1 kgf/m^2 = 0.2048 lbf/ft^2 = 9.80665 N/m^2
1 lbf/ft^2 = 4.8824 kgf/m^2 = 47.88 N/m^2

F. Moment

1 N-m = 8.851 lbf-in. = 0.7376 lbf-ft
1 ft-lbf = 1.356 N-m

G. Velocity

1 kilometer/hr (km/hr) = 0.621 miles/hr (MPH)
1 MPH = 1.609 km/hr

7.0 REFERENCES

7.1 General References

A. Additional Design Sources

(a) Structural Engineering Handbook (1979 Edition)
Gaylord and Gaylord
McGraw-Hill Book Company
New York, New York

(b) Wind Loading on Buildings (1975 Edition)
Angus J. MacDonald
Applied Science Publishers, Ltd.
London, England

B. Other Sources

(a) Thermal Environmental Engineering (1970 Edition)
James L. Threlkeld
Prentice-Hall, Inc.
Englewood Cliffs, New Jersey

(b) Solar Energy Thermal Process (1974 Edition)
John A. Duffie and William A. Beckman
A Wiley-Interscience Publication
John Wiley and Sons, Inc.
New York - London - Sydney - Toronto

7.2 Applicable Industry Codes and Standards

A. Codes: See list in Section 4.2.

B. Material Standards

(a) Manual of Steel Construction (8th Edition)
American Institute of Steel Construction, Inc.
101 Park Avenue
New York, NY 10017

(b) Timber Construction Manual (2nd Edition, 1974)
American Institute of Timber Construction
333 West Hampden Avenue
Englewood, Colorado 80110

(c) CRSI Handbook (1980 Edition)
Concrete Reinforcing Steel Institute
180 North LaSalle Street
Chicago, Illinois 60601

(d) Cold-Formed Steel Design Manual (1977 Edition)
American Iron and Steel Institute
1000 16th Street NW
Washington, D.C. 20036

(e) Handbook of Plastics and Elastomers (1975 Edition)
Charles A. Harper, Editor-in-Chief
McGraw-Hill Book Company
New York, New York

(f) Manual of Cold Form Welded Structural Steel Tubing (1st Edition)
Welded Steel Tube Institute, Inc.
Structural Tube Division
522 Westgate Tower
Cleveland, Ohio 44116

(g) Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI-315-74, 1974 Edition)
Manual of Concrete Practice (1977 Edition)
American Concrete Institute
Box 19150, Redford Station
Detroit, Michigan 48219

(h) Welding Aluminum (1972 Edition)
American Welding Society
2501 NW 7th Street
Pittsburgh, Pennsylvania

(i) Aluminum Standards and Data (1979 Edition)
Specifications for Aluminum Structures (1976 Edition)
Engineering Data for Aluminum Structures (1975 Edition)
The Aluminum Association, Inc.
818 Connecticut Avenue NW
Washington, D.C. 20006

7.3 List of Manufacturers and Suppliers of Equipment Components and Materials

7.3.1 Materials

A. Steel: American Institute of Steel Construction, Inc. (New York, New York)
B. Wood: American Institute of Timber Construction (Englewood, Colorado)
C. Cold-Formed Steel: American Iron and Steel Institute (Washington, D.C.)
D. Aluminum: The Aluminum Association (Washington, D.C.).
E. Concrete: Concrete Reinforcing Steel Institute (Chicago, Illinois)

7.3.2 Mechanical Components

The following is a partial list of mechanical component manufacturers.

A. Columbia Nut and Bolt Corp. (New York, New York)
B. Fasteners and Metal Products Corp. (Waltham, Massachusetts)
C. H-P Products (Louisville, Ohio)
D. Jarvis Steel & Lumber Co., Inc. (Baltimore, Maryland)
E. Kee Klamp (Buffalo, New York)
F. P.G. Structures, Inc. (New York, New York)
G. Unistrut (Baltimore, Maryland)

7.3.3 Electrical Components

The following is a partial list of electrical component manufacturers.

A. Air Born Connectors (Columbia, Maryland)
B. Ampheno 1 (Bunker Ramo) (Broadview, Illinois)
C. Brad Harrison (La Grange, Illinois)
D. Camolok (Cincinnati, Ohio)
E. General Connector Corporation (Newton, Massachusetts)
F. ITT Cannon (Santa Ana, California)
G. Winchester Electronics (Oakville, Connecticut)

7.3.4 Modules

A. Amperex Electronic Corporation (Slatesville, Rhode Island)
B. ARCO Solar (Chatsworth, California)
C. Mobil Tyco Solar Energy (Waltham, Massachusetts)
D. Motorola Solar Systems (Phoenix, Arizona)
E. Optical Coating Laboratories, Inc. (OCLI) (City of Industry, California)
F. SES (Newark, Delaware)
G. Sensor Technology (Chatsworth, California.)
H. Silicon Material, Inc. (Mantain View, California)
I. Silicon Sensor, Inc. (Dodgeville, Wisconsin)
J. Solar Power Corp (North Billercia, Massachusetts)
K. Solarex (Rockville, Maryland)
L. Solenergy Corp. (Wakefield, Massachusetts)
M. Solec International (Los Angeles, California)
N. Sollos, Inc. (West Los Angeles, California)
O. Tideland Signal (Houston, Texas)
P. Texas Instruments (Dallas, Texas)

7.4 Wind Load Design Criteria

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Englewood, Colorado 80110

WIND LOADS

General

Designing structures to resist wind loading is, like the analysis for snow loading, a very complex engineering problem. Considerable research has been conducted to evaluate wind effects on various structures that has resulted in the establishment of design pressure coefficients that account for building shape and wind direction. In addition, extensive studies of basic wind velocities related to geographical location have resulted in the development of detailed wind velocity maps for the United States. Other studies of surface resistance relative to the degree of land development and gust characteristics at a given location have provided a method for a further refinement of the basic wind velocity and its effect on structures. However, much work remains to be done to relate further the dynamic behavior of structures to wind forces that are attributable to gusting and turbulence.

The wind load analysis information presented here is intended to provide the engineer with a design procedure that accounts for the basic parameters affecting wind loading of structures. The material has been compiled from a variety of sources, including the *American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*, American National Standards Institute; *Wind Forces on Structures*, Paper No. 3269, Final Report of the Task Committee on Wind Forces of the Committee on Loads and Stresses of the Structural Division, ASCE; *New Distributions of Extreme Winds in the United States* by H. C. S. Thom, Vol. 94, ST 7, July 1968, *Journal of the Structural Division*, ASCE; *Strength of Houses*, Building Materials and Structures Report 109, National Bureau of Standards, U.S. Department of Commerce; and *Structural Information for Building Design in Canada*, 1965, Supplement No. 3 to the National Building Code of Canada.

Basic Wind Velocities

Figures 3.2 and 3.3 represent wind probability maps for 50- and 100-year mean recurrence intervals, respectively. These figures provide basic wind velocities for observed air flows in open, level country at a height of 30 ft above the ground. For the design of most permanent structures, a basic wind speed with a 50-year mean recurrence interval should be applied. However, if in the judgment of the engineer or authority having jurisdiction, the structure presents an unusually high degree of hazard to life and property in case of failure, a 100-year mean recurrence interval wind velocity should be used for design. Similarly, for temporary structures or structures having negligible risk of human life in case of failure, a design wind velocity based on a 25-year mean recurrence interval may be used. Additional wind velocity maps for various mean recurrence intervals are given in *New Distribution of Extreme Winds in the United States*, by H. C. S. Thom, Vol. 94, ST 7, July 1968, *Journal of the Structural Division*, ASCE.

Since the wind velocities given in Figures 3.2 and 3.3 are for a height of 30 ft, it is necessary to modify this value for other design heights. An accepted procedure is to apply an exponential formula of the form

$$V_h = V_{30}\left(\frac{h}{30}\right)^{1/x}$$

where V_h = wind velocity at any height
V_{30} = wind velocity at a height of 30 ft
h = height
x = exponent depending upon general site exposure conditions such as follows:

For level or slightly rolling terrain with minimal obstructions such as airports, x may be taken as 7; for rolling terrain with numerous obstructions such as suburban areas, a value of 5 may be assumed; and for urban areas, a value of 3 is recommended.

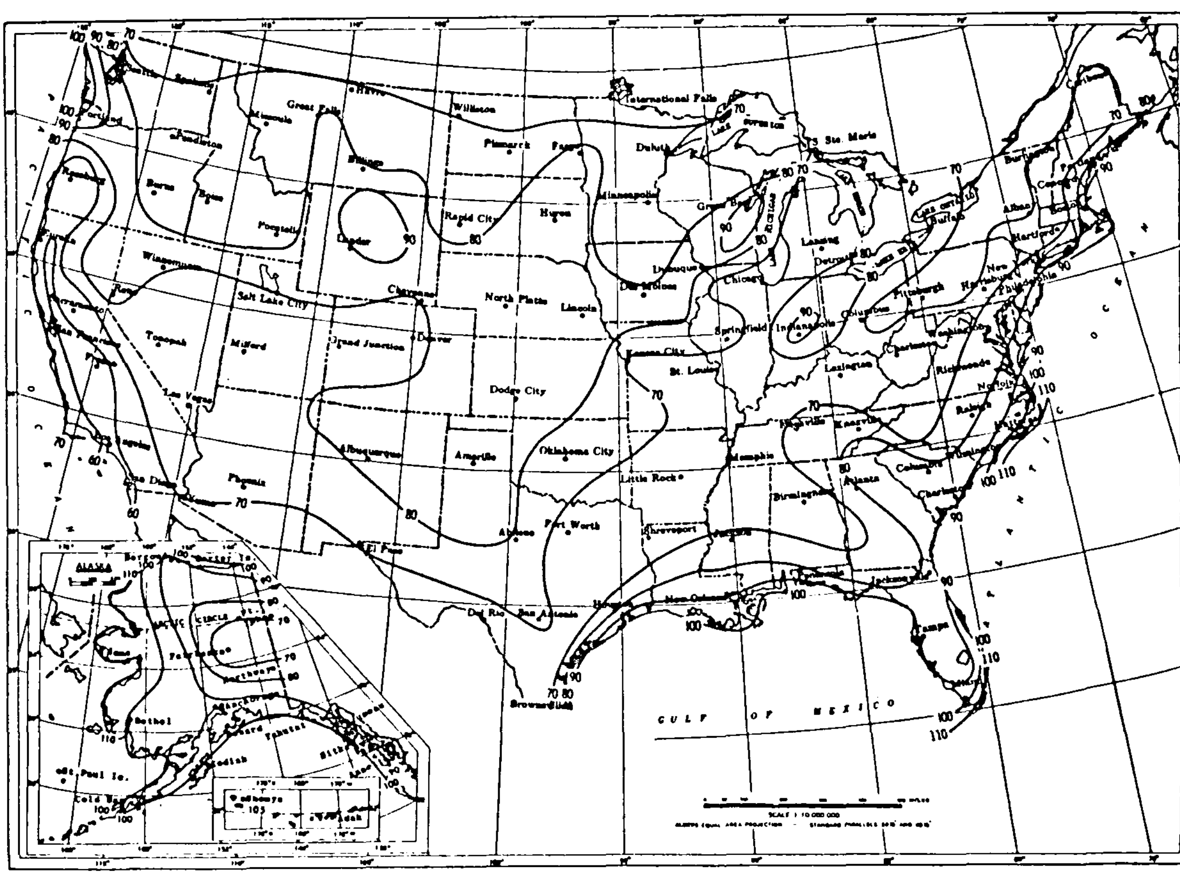

Figure 3.2. ISOTACH 0.02 QUANTILES, IN MPH: ANNUAL EXTREME-MILE 30 FT ABOVE GROUND, 50-YR MEAN RECURRENCE INTERVAL. Source: *New Distribution of Extreme Winds in the United States*, by H. C. S. Thom, Paper No. 6038, Vol. 94, ST7, July 1968, *Journal of the Structural Division*, ASCE.

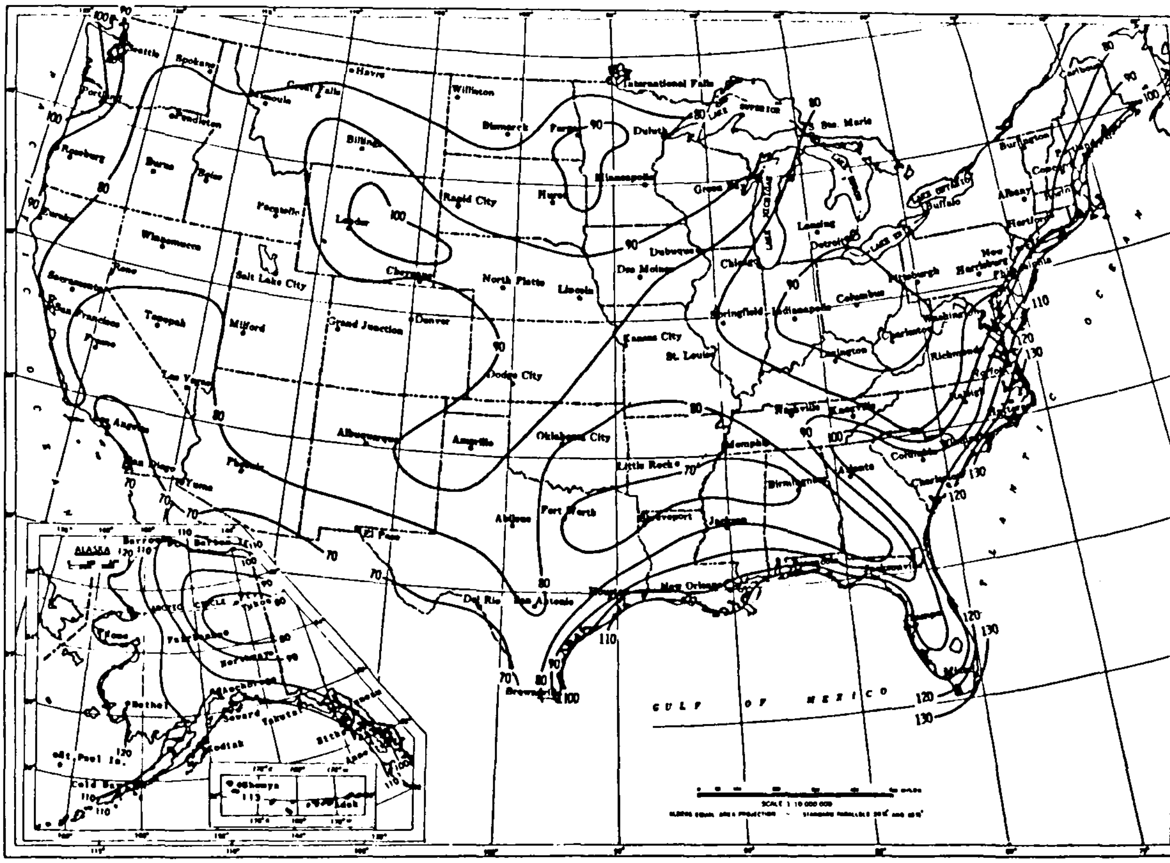

Figure 3.3. ISOTACH 0.01 QUANTILES, IN MPH: ANNUAL EXTREME-MILE 30 FT ABOVE GROUND, 100-YR MEAN RECURRENCE INTERVAL. Source: *New Distribution of Extreme Winds in the United States*, by H. C. S. Thom, Paper No. 6038, Vol. 94, ST7, July 1968, *Journal of the Structural Division*, ASCE.

Thus for open, relatively level areas similar to those from which the data given in Figures 3.2 and 3.3 are derived, the following equation is applicable:

$$V_h = V_{30}\left(\frac{h}{30}\right)^{1/7}$$

This is commonly referred to as the $\frac{1}{7}$th power law for determining wind velocities.

Based on the use of this relationship, Table 3.6 gives wind velocities for various height zones based on multiplying the basic wind velocity obtained from Figure 3.2 or 3.3 by an average correction factor for that height zone.

TABLE 3.6

WIND VELOCITIES CORRESPONDING TO VARIOUS HEIGHTS

Height, h, ft	Approximate Height Zone	Correction Factor[a]	Basic Wind Velocities, mph					
			60	70	80	90	100	110
30	20 to 40 ft	1.00	60	70	80	90	100	110
60	41 to 80 ft	1.10	66	77	88	99	110	121
110	81 to 140 ft	1.20	72	84	96	108	120	132
190	141 to 240 ft	1.30	78	91	104	117	130	143
320	241 to 400 ft	1.40	84	98	112	126	140	154

[a]Based on the relationship $V_h = V_{30}(h/30)^{1/7}$.

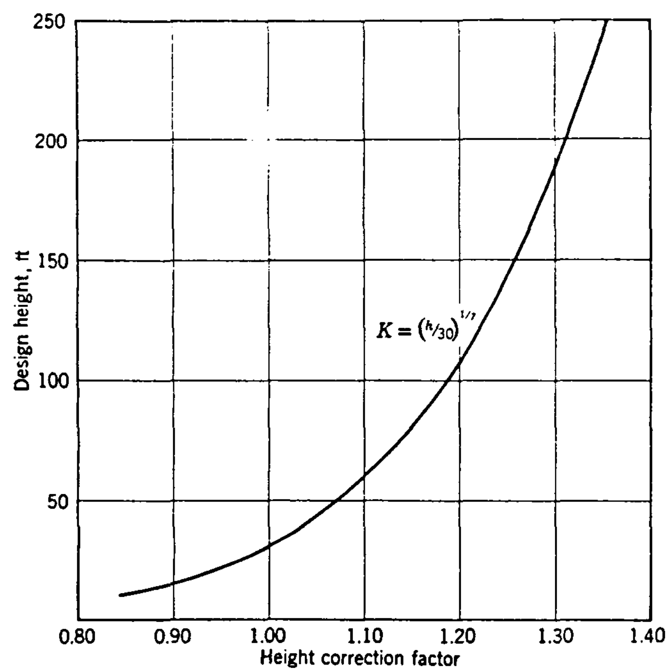

Figure 3.4. HEIGHT CORRECTION FACTOR BASED ON 1/7th POWER LAW.

Similarly, Figure 3.4 can be used to determine a height correction factor based on the $\frac{1}{7}$th power law for any specific height to be analyzed in design.

In addition to adjusting the basic wind velocities for height and site exposure conditions, it is also possible to apply a gust response factor in the determination of design velocity pressures. The concept of determining gust response factors is very complex and the engineer is referred to the following ASCE papers for a detailed analysis of gust loading of structures: *Gust Loading Factors* by Davenport, Vol. 93, ST 3, June 1967, *Journal of the Structural Division*, and *Gust Response Factors* by Vellozzi and Cohen, Vol, 94, ST 6, June 1968, *Journal of the Structural Division*.

As a general guide, the gust response factor is primarily a function of the size and height of the structure and the surface roughness and obstructions existing in the surrounding area. For small-to-medium size structures located in open, relatively level terrain and ranging in height up to approximately 100 ft, a gust response factor of 1.3 is commonly assumed. Similarly, for taller buildings of approximately 400 ft in height, a gust response factor of 1.1 can be used for design purposes.

It is recommended that the designer use a dynamic analysis accounting for wind turbulence and the size and natural frequency of the structure to determine gusting effects on structures greater than 400 ft in height. Therefore, a design wind velocity is determined by modifying the basic wind velocity obtained from either Figure 3.2 or 3.3 for height, site exposure, and the gust effect. For additional information related to the effect of site exposure on wind velocity, the engineer is referred to the *American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*, ANSI A 58.1-1972.

Velocity Pressures

For standard air (0.07651 pcf, corresponding to 15°C at 760 mm of mercury) and velocity of wind, V, expressed in miles per hour, the velocity (dynamic) pressure, q, in psf, is given by

$$q = 0.00256V^2$$

Thus the dynamic pressure for any given combination of geographic location, height of structure, and basic wind velocity can be determined by using the design wind velocity in the above equation. Table 3.7 gives velocity pressure values for various heights and basic wind velocities for a site located in open, level terrain. The gust response factor has not been incorporated in this table but can easily be accounted for in a separate calculation by multiplying the tabulated values by the applicable gust response factor.

Therefore, for general design purposes, the effective velocity pressure for any height, h, may be determined from the relationship

$$q_h = (0.00256)(V_{30})^2(h/30)^{2/x}(G_F)$$

where V_{30} = basic wind velocity obtained from Figure 3.2 or 3.3
h = height of building
x = exponent depending upon general site exposure conditions
G_F = gust response factor

TABLE 3.7

VELOCITY PRESSURES,[a] q, CORRESPONDING TO BASIC WIND VELOCITIES, V

Height,[b] ft	Basic Wind Velocity, mph[c]					
	60	70	80	90	100	110
30	9.2	12.5	16.4	20.7	25.6	31.0
60	11.2	15.2	20.0	25.2	31.2	37.8
110	13.3	18.1	23.8	30.0	37.1	44.9
190	15.6	21.2	27.8	35.1	43.4	52.5
320	18.1	24.6	32.3	40.8	50.4	61.1
510	20.7	28.1	36.9	46.6	57.6	69.7

[a] Velocity pressure based on $q_{30} = 0.00256V_{30}^2$.
[b] Height correction based on $q_h = q_{30}(h/30)^{2/7}$.
[c] From Figure 3.2 or 3.3.

Example. Calculate the effective velocity pressure acting on a permanent structure to be located in relatively open, level terrain near Cheyenne, Wyoming. Assume the design height to be 100 ft and that a 50-year mean recurrence interval wind velocity is applicable.

From Fig. 3.2, a basic wind velocity of 80 mph for a height of 30 ft is applicable for Cheyenne, Wyoming.

Since the height of the structure is 100 ft, and the structure is to be located in relatively open, level terrain, a gust response factor of 1.3 is applied.

Also for this type of site exposure it is recommended that an exponential value of x be taken as 7. The effective velocity pressure for this condition is:

$$q_{100} = 0.00256(80)^2\left(\frac{100}{30}\right)^{2/7}(1.3)$$

$$q_{100} = 30.0 \text{ psf}$$

7.5 Wind Load Data

The information in this section was reproduced from the U.S. Army and Air Force Manual TM5-809-1/AFM 88-3 with permission of the U.S. Army Corps of Engineers.

Note: When available, the American National Standards Institute's (ANSI's) Building Code Requirements for Minimum Design Loads in Buildings and Other Structures (A58-1) should be used instead of the "Peak Velocity" data given in these tables.

TABLE II. WIND, SNOW, AND FROST DATA

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
CONTIGUOUS UNITED STATES:			
ALABAMA			
Brookley AFB	120	20	6
Maxwell AFB	90	20	9
Mobile	120	20	6
Montgomery	90	20	6
ARIZONA			
Davis Monthan AFB	80	20	5
Luke AFB	90	20	7
Williams AFB	80	20	7
Phoenix	80	20	7
ARKANSAS			
Little Rock AFB	90	15	12
CALIFORNIA			
Castle AFB	80	20	5
Hamilton AFB	85	20	5
March AFB	80	20	5
Mather AFB	100	20	5
Travis AFB	80	20	5
Vandenberg AFB	80	20	5
San Diego	80	20	0
Pasadena	80	20	0
Long Beach	80	20	0
San Francisco	85	20	5
Oakland	85	20	5
Mare Island	85	20	5
Sacramento	105	20	5
Stockton	90	20	5
COLORADO			
Lowry AFB	80	20	60
Denver	80	20	60

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
CONNECTICUT			
New London	80	20	35
New Haven	80	20	35
DELAWARE			
Dover AFB	95	20	20
FLORIDA			
Eglin AFB	125	20	5
Homestead AFB	125	20	0
MacDill AFB	90	20	2
Patrick AFB	125	20	2
Jacksonville	105	20	2
Miami	125	20	0
Key West	125	20	0
Pensacola	125	20	2
Tampa	90	20	2
GEORGIA			
Hunter AFB	105	20	5
Robins AFB	80	20	5
Turner AFB	85	20	5
Augusta	85	20	5
Atlanta	85	20	7
Savannah	105	20	3
Macon	85	20	5
IDAHO			
Mountain Home AFB	85	20	40
ILLINOIS			
Chanute AFB	95	20	35
Scott AFB	80	20	35
Chicago	85	20	40

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
INDIANA			
Fort Wayne	90	20	40
Indianapolis	105	20	30
IOWA			
Sioux City	100	20	54
KANSAS			
Forbes AFB	110	20	30
Schilling AFB	100	20	24
KENTUCKY			
Lexington	90	20	18
Louisville	90	20	18
LOUISIANA			
Barksdale AFB	80	20	5
Chennault AFB	120	20	4
New Orleans	120	20	2
MAINE			
Dow AFB	100	40	75
Loring AFB	90	40	75
Portland	100	40	65
Bangor	100	40	72
MARYLAND			
Andrews AFB	90	20	25
Baltimore	90	20	22
MASSACHUSETTS			
L. G. Hanscom Field	110	25	50
Otis AFB	120	20	50
Westover AFB	85	30	70
Boston	110	25	50
Springfield	85	30	70

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
MICHIGAN			
Kinchelow AFB	95	40	65
Selfridge AFB	80	20	50
Detroit	80	20	50
MINNESOTA			
Minn-St.Paul IAP	90	35	75
Minneapolis	90	35	75
Duluth	100	40	75
MISSISSIPPI			
Jackson	105	20	3
MISSOURI			
Kansas City	100	20	28
St. Louis	80	20	27
MONTANA			
Malmstrom AFB	85	25	75
NEBRASKA			
Offutt AFB	95	25	55
Omaha	95	25	55
NEVADA			
Nellis AFB	90	20	8
Stead AFB	90	25	23
Fallon	90	25	12
Hawthorne	90	25	30
Reno	95	25	23
NEW HAMPSHIRE			
Pease AFB	105	30	60
Portsmouth	105	30	60

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
NEW JERSEY			
McGuire AFB	85	20	30
Atlantic City	100	20	20
Bayonne	85	20	30
NEW MEXICO			
Cannon AFB	80	20	15
Holloman AFB	80	20	20
Walker AFB	85	20	15
Albuquerque	100	20	17
NEW YORK			
Griffis AFB	80	40	50
Plattsburg AFB	90	35	70
Stewart AFB	90	25	45
Buffalo	90	30	35
Albany	80	30	54
New York	85	20	40
Syracuse	80	40	56
NORTH CAROLINA			
Pope AFB	80	20	9
Charlotte	90	20	8
Wilmington	130	20	5
NORTH DAKOTA			
Grand Forks AFB	100	25	85
Minot AFB	100	20	80
OHIO			
Wright-Patterson AFB	90	20	40
Columbus	90	20	40
Cincinnati	90	20	20
OKLAHOMA			
Tinker AFB	90	20	20

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
OREGON			
Portland Int. Apt.	115	20	6
Portland	115	20	6
PENNSYLVANIA			
Olmsted AFB	80	20	35
Harrisburg	85	20	30
Pittsburg	85	20	38
Philadelphia	80	20	30
RHODE ISLAND			
Providence	115	20	45
SOUTH CAROLINA			
Shaw AFB	80	20	6
Charleston	120	20	3
SOUTH DAKOTA			
Ellsworth AFB	105	20	55
TENNESSEE			
Sewart AFB	95	20	10
Memphis	90	20	10
TEXAS			
Amarillo AFB	120	20	20
Bergstrom AFB	85	20	4
Biggs AFB	90	20	6
Carswell AFB	85	20	12
Dyess AFB	100	20	10
Ellington AFB	90	20	3
Kelly AFB	90	20	4
Reese AFB	85	20	15
Sheppard AFB	85	20	15
Corpus Christi	100	20	2
El Paso	90	20	6
Fort Worth	80	20	10
Galveston	100	20	3
Houston	90	20	3
San Antonio	80	20	4
Amarillo	120	20	20

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
UTAH			
Hill AFB	95	30	35
Salt Lake City	90	25	35
VERMONT			
Burlington	90	35	72
VIRGINIA			
Langley AFB	110	20	6
Newport News	105	20	10
Norfolk	105	20	10
Richmond	90	20	14
WASHINGTON			
Fairchild AFB	90	25	65
Larson AFB	80	25	35
McChord AFB	85	20	10
Bremerton	85	20	9
Seattle	85	20	8
Spokane	90	20	30
Pasco	80	30	25
Tacoma	85	20	8
WEST VIRGINIA			
Charleston	80	20	30
WISCONSIN			
Truax Field	115	25	50
Milwaukee	110	25	54
Green Bay	100	25	54
WYOMING			
Francis E. Warren AFB	100	20	70
WASHINGTON, DC	90	20	20

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
AFRICA:			
Libya			
Wheelus AB	85	20	0
Morocco			
Casablanca	85	20	0
Port Lyautey NAS	85	20	0
ASIA:			
India			
Bombay	85	20	0
Calcutta	105	20	0
Madras	85	20	0
New Delhi	85	20	0
Japan			
Itazuke AB	90	20	6
Johnson AB	105	20	6
Misawa AB	95	20	18
Tachikawa AB	100	20	6
Tokyo	100	20	6
Wakkanai	115	40	36
Korea			
Kimpo AB	80	20	30
Seoul	80	20	30
Uijongbu	80	20	36
Pakistan			
Peshawar	80	20	6
Saudia Arabia			
Bahrain Island	80	20	0
Dhahran AB	80	20	0
Taiwan			
Tainan	120	20	0
Taipei	130	20	0
Thailand			
Bangkok	80	20	0
Turkey			
Ankara	90	20	24
Karamursel	105	20	12
Viet Nam			
Saigon	95	20	0

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
ATLANTIC OCEAN AREA:			
Ascension Island	80	20	0
Azores			
Lajes Field	115	20	0
Bermuda			
Bermuda NAS	110	20	0
Kindley AFB	125	20	0
CARIBBEAN SEA:			
Bahama Islands			
Eleuthera Island	140	20	0
Grand Bahama Island	140	20	0
Great Exuma Island	140	20	0
Cuba			
Guantanamo NAS	80	20	0
Leeward Islands			
Antigua Island	140	20	0
Puerto Rico			
Raney AFB	95	20	0
San Juan	115	20	0
Viequest Island	140	20	0
Trinidad Island			
Port of Spain	80	20	0
Trinidad NS	80	20	0
CENTRAL AMERICA:			
Canal Zone			
Albrook AFB	80	20	0
Balboa	80	20	0
Coco Solo	80	20	0
Colon	80	20	0
Cristobal	80	20	0
France AFB	80	20	0
EUROPE:			
England			
Birmingham	85	20	12
London	90	20	12
Mildenhall AB	95	20	12

Table II (Continued)

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
EUROPE:			
England			
Plymouth	85	20	12
Sculthorpe AB	90	20	12
Southport	95	20	12
South Shields	90	20	12
Spurn Head	90	20	12
France			
Nancy	80	20	18
Paris/Le Bourget	95	20	18
Rennes	100	20	18
Vichy	115	25	24
Germany			
Bremen	80	25	30
Munich-Riem	90	40	36
Rhein-Main AB	80	25	30
Stuttgart AB	85	40	36
Greece			
Athens	85	20	0
Italy			
Aviano AB	80	20	18
Brindisi	100	20	6
Scotland			
Aberdeen	85	20	12
Edinburgh	90	20	12
Edzell	85	20	12
Glasgow/Renfrew Airfield	95	20	12
Lerwick, Shetland Islands	105	20	18
Londonderry	125	20	12
Prestwick	95	20	12
Stornoway	110	20	12
Thurso	100	20	12
Spain			
Madrid	80	20	6
Rota	85	20	0
San Pablo	110	20	6
Zaragoza	110	20	6

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
NORTH AMERICA:			
Alaska			
Adak, Aleutian Is.	125	20	24
Anchorage	95	35	60
Annette	95	20	24
Attu	180	35	24
Barrow	100	20	Permafrost
Bethel	95	35	60
Cold Bay	110	20	36
Cordova	95	40	48
Eielson AFB	80	35	60
Elmendorf AFB	95	35	60
Fairbanks	80	35	60
Gambell	130	25	48
Juneau	90	40	36
King Salmon	115	20	60
Kodiak	115	20	48
Kotzebue	120	20	Permafrost
McGrath	85	40	84
Middleton Island AFS	125	40	48
Nikolaski, Umnak Is.	130	25	36
Nome	120	40	Permafrost
Northeast Cape AFS, St. Lawrence Is.	135	25	48
Shemya Island	180	35	24
St. Paul Island	105	25	36
Umiat	110	30	Permafrost
Wales	105	40	Permafrost
Yakutat	100	40	36
Canada			
Argentia NAS, Nfld.	105	40	36
Churchill, Manitoba	100	40	Permafrost
Cold Lake, Alberta	80	40	72
Edmonton, Alberta	80	30	60
E. Harmon AFB, NFld.	105	40	60
Fort William, Ontario	80	40	60
Frobisher, N.W.T.	100	40	Permafrost
Goose Airport, Nfld.	85	40	60
Ottawa, Ontario	85	40	48
St. John's, Nfld.	105	40	36
Toronto, Ontario	85	40	36
Winnipeg, Manitoba	80	40	60

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
NORTH AMERICA:			
Greenland			
Narsarssuak AB	130	30	60
Simiutak AB	155	25	60
Sondrestrom AB	110	20	Permafrost
Thule AB	130	25	Permafrost
Iceland			
Keflavik	115	30	24
Thorshofn	135	30	36
PACIFIC OCEAN AREA:			
Caroline Islands			
Koror, Palau Islands	95	20	0
Ponape	110	20	0
Hawaii			
Barber's Point	80	20	0
Hickam AFB	80	20	0
Kaneohe Bay	85	20	0
Wheeler AFB	80	20	0
Johnston Island	80	20	0
Mariana Islands			
Agana, Guam	155	20	0
Andersen AFB, Guam	155	20	0
Saipan	150	20	0
Tinian	150	20	0
Marshall Islands			
Kwajalein	105	20	0
Eniwetok	105	20	0
Marcus Island	150	20	0
Midway Island	85	20	0
Okinawa			
Kadena AB	185	20	0
Naha AB	180	20	0
Philippine Islands			
Clark AFB	85	20	0
Sangley Point	80	20	0
Subic Bay	80	20	0

Table II (Continued).

Location	Peak Velocity (Gust included) MPH	Roof Snow or Live Load PSF	Max. Frost Penetration Inches
PACIFIC OCEAN AREA:			
Samoa Islands			
Apia, Upolu Island	145	20	0
Tutuila, Tutuila Island	145	20	0
Volcano Islands			
Iwo Jima AB	205	20	0
Wake Island	85	20	0

7.6 Seismic Load Design

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for these individual members).

(h) **Miscellaneous Structures.** Greenhouses, lath houses and agricultural buildings shall be designed for the horizontal wind pressures as set forth in Table No. 23-F, except that, if the height zone is 20 feet or less, two-thirds of the first line of listed values may be used. The structures shall be designed to withstand an uplift wind pressure equal to three-fourths of the horizontal pressure.

(i) **Moment of Stability.** The overturning moment calculated from the wind pressure shall in no case exceed two-thirds of the dead load resisting moment.

The weight of earth superimposed over footings may be used to calculate the dead load resisting moment.

(j) **Combined Wind and Live Loads.** For the purpose of determining stresses all vertical design loads except the roof live load and crane loads shall be considered as acting simultaneously with the wind pressure.

> **EXCEPTION:** Where snow loading is required in the design of roofs, at least 50 percent of such snow load shall be considered acting in combination with the wind load. The Building Official may require that a greater percentage of snow load be considered due to local conditions.

Earthquake Regulations

Sec. 2312. (a) **General.** Every building or structure and every portion thereof shall be designed and constructed to resist stresses produced by lateral forces as provided in this Section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the base. The force shall be assumed to come from any horizontal direction.

Structural concepts other than set forth in this Section may be approved by the Building Official when evidence is submitted showing that equivalent ductility and energy absorption are provided.

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

(b) **Definitions.** The following definitions apply only to the provisions of this Section:

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

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

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

DUCTILE MOMENT RESISTING SPACE FRAME is a moment resisting space frame complying with the requirements for a ductile moment resisting space frame as given in Section 2312 (j).

ESSENTIAL FACILITIES—See Section 2312 (k).

LATERAL FORCE RESISTING SYSTEM is that part of the structural system assigned to resist the lateral forces prescribed in Section 2312 (d) 1.

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

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

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

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

(c) **Symbols and Notations.** The following symbols and notations apply only to the provisions of this Section:

C = Numerical coefficient as specified in Section 2312 (d) 1.

C_p = Numerical coefficient as specified in Section 2312 (g) and as set forth in Table No. 23-J.

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

δ_i δ_n = Deflections at levels i and n respectively, relative to the base, due to applied lateral forces or as determined in Section 2312 (h).

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

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

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

g = Acceleration due to gravity.

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

I = Occupancy Importance Factor as specified in Table No. 23-K.

K = Numerical coefficient as set forth in Table No. 23-I.

Level i

I = Level of the structure referred to by the subscript i.

i = 1 designates the first level above the base.

Level n

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

Level x

= That level which is under design consideration.

x = 1 designates the first level above the base.

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

S = Numerical coefficient for site-structure resonance.

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

T_s = Characteristic site period.

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

W = The total dead load as defined in Section 2302 including the partition loading specified in Section 2304 (d) where applicable.

EXCEPTION: "W" shall be equal to the total dead load plus 25 percent of the floor live load in storage and warehouse occupancies. Where the design snow load is 30 psf or less, no part need be included in the value of "*W*." Where the snow load is greater than 30 psf, the snow load shall be included; however, where the snow load duration warrants, the Building Official may allow the snow load to be reduced up to 75 percent.

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

W_p = The weight of a portion of a structure.

Z = Numerical coefficient dependent upon the zone as determined by Figures No. 1, No. 2 and No. 3 in this Chapter. For locations in Zone No. 1, $Z = \frac{3}{16}$. For locations in Zone No. 2, $Z = \frac{3}{8}$. For locations in Zone No. 3, $Z = \frac{3}{4}$. For locations in Zone No. 4, $Z = 1$.

(d) **Minimum Earthquake Forces for Structures.** Except as provided in Section 2312 (g) and (i), every structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = ZIKCSW \quad \text{(12-1)}$$

The value of K shall be not less than that set forth in Table No. 23-I. The value of C and S are as indicated hereafter except that the product of CS need not exceed 0.14.

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

$$C = \frac{1}{15\sqrt{T}} \quad \text{(12-2)}$$

The value of C need not exceed 0.12.

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

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\delta_i^2\right) \div g\left[\sum_{i=1}^{n-1} F_i\delta_i + (F_t + F_n)\delta_n\right]} \quad \text{(12-3)}$$

where the values of F_i, F_t, δ_i and δ_n shall be determined from the base shear V, distributed approximately in accordance with the principles of Formulas (12-5), (12-6) and (12-7) or any arbitrary base shear with a rational distribution.

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

$$T = \frac{0.05h_n}{\sqrt{D}} \quad \text{(12-3A)}$$

Or in buildings in which the lateral force resisting system consists of ductile moment-resisting space frames capable of resisting 100 percent of the required lateral forces and such system is not enclosed by or adjoined by more rigid elements tending to prevent the frame from resisting lateral forces:

$$T = 0.10N \qquad (12\text{-}3B)$$

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

For $T/T_s = 1.0$ or less $\quad S = 1.0 + \frac{T}{T_s} - 0.5\left[\frac{T}{T_s}\right]^2$

$$\qquad (12\text{-}4)$$

For T/T_s greater than 1.0 $\quad S = 1.2 + 0.6\frac{T}{T_s} - 0.3\left[\frac{T}{T_s}\right]^2$

$$\qquad (12\text{-}4A)$$

WHERE:

T in Formulas (12-4) and (12-4A) shall be established by a properly substantiated analysis but T shall be not less than 0.3 second.

The range of values of T_s may be established from properly substantiated geotechnical data, in accordance with U.B.C. Standard No. 23-1, except that T_s shall not be taken as less than 0.5 second nor more than 2.5 seconds. T_s shall be that value within the range of site periods, as determined above, that is nearest to T.

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

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

(e) **Distribution of Lateral Forces.** 1. **Structures having regular shapes or framing systems.** The total lateral force V shall be distributed over the height of the structure in accordance with Formulas (12-5), (12-6) and (12-7).

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (12\text{-}5)$$

The concentrated force at the top shall be determined according to the following formula:

$$F_t = 0.07TV \qquad (12\text{-}6)$$

F_t need not exceed $0.25V$ and may be considered as 0 where T is 0.7 second or less. The remaining portion of the total base shear V shall be distributed over the height of the structure including level n according to the following formula:

$$F_x = \frac{(V - F_t)\, w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \text{(12-7)}$$

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

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

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

4. **Distribution of horizontal shear.** Total shear in any horizontal plane shall be distributed to the various elements of the lateral force resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm.

Rigid elements that are assumed not to be part of the lateral force resisting system may be incorporated into buildings provided that their effect on the action of the system is considered and provided for in the design.

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

(f) **Overturning.** Every building or structure shall be designed to resist the overturning effects caused by the wind forces and related requirements specified in Section 2311, or the earthquake forces specified in this Section, whichever governs.

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

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

(g) **Lateral Force on Elements of Structures.** Parts or portions of structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = ZIC_pSW_p \quad \text{. . . . . (12-8)}$$

EXCEPTION: Where C_p in Table No. 23-J is 1.0 or more the value of I and S need not exceed 1.0.

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

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

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

(i) **Alternate Determination and Distribution of Seismic Forces.** Nothing in Section 2312 shall be deemed to prohibit the submission of properly substantiated technical data for establishing the lateral forces and distribution by dynamic analyses, in such analyses the dynamic characteristics of the structure must be considered.

(j) **Structural Systems. 1. Ductility requirements.** A. All buildings designed with a horizontal force factor $K = 0.67$ or 0.80 shall have ductile moment resisting space frames.

B. Buildings more than 160 feet in height shall have ductile moment resisting space frames capable of resisting not less than 25 percent of the required seismic forces for the structure as a whole.

EXCEPTION: Buildings more than 160 feet in height in Seismic Zone No. 1 may have concrete shear walls designed in conformance with Section 2627 of this Code in lieu of a ductile moment resisting space frame, provided a K value of 1.00 or 1.33 is utilized in the design.

C. In Seismic Zones No. 2, No. 3 and No. 4 all concrete space frames required by design to be part of the lateral force resisting system and all concrete frames located in the perimeter line of vertical support shall be ductile moment resisting space frames.

EXCEPTION: Frames in the perimeter line of the vertical support of buildings designed with shear walls taking 100 percent of the design lateral forces need only conform with Section 2312 (j) 1D.

D. In Seismic Zones No. 2, No. 3 and No. 4 all framing elements not required by design to be part of the lateral force resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity and induced moment due to $3/K$ times the distortions resulting from the Code required lateral forces. The rigidity of other elements shall be considered in accordance with Section 2312 (e) 4.

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

F. The necessary ductility for a ductile moment resisting space frame shall be provided by a frame of structural steel with moment resisting connections (complying with Section 2722 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or Section 2723 for buildings in Seismic Zone No. 1) or by a reinforced concrete frame (complying with Section 2626 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or Section 2625 for buildings in Seismic Zone No. 1).

G. In Seismic Zones No. 2, No. 3 and No. 4 all members in braced frames shall be designed for 1.25 times the force determined in accordance with Section 2312 (d). Connections shall be designed to develop the full capacity of the members or shall be based on the above forces without the one-third increase usually permitted for stresses resulting from earthquake forces.

Braced frames in buildings shall be composed of axially loaded bracing members of A36, A440, A441, A501, A572 (except Grades 60 and 65) or A588 structural steel; or reinforced concrete members conforming to the requirements of Section 2627.

H. Reinforced concrete shear walls for all buildings shall conform to the requirements of Section 2627.

I. In structures where $K = 0.67$ and $K = 0.80$, the special ductility requirements of structural steel (complying with Section 2722 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or Section 2723 for buildings in Seismic Zone No. 1) or by reinforced concrete (complying with Section 2626 for buildings in Seismic Zones No. 2, No. 3 and No. 4 or with Section 2625 for buildings in Seismic Zone No. 1), as appropriate, shall apply to all structural elements below the base which are required to transmit to the foundation the forces resulting from lateral loads.

2. Design requirements. A. **Minor alterations.** Minor structural alterations may be made in existing buildings and other structures, but the resistance to lateral forces shall be not less than that before such alterations were made, unless the building as altered meets the requirements of this Section.

B. **Reinforced masonry or concrete.** All elements within structures located in Seismic Zones No. 2, No. 3 and No. 4 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete under the provisions of Chapters 24 and 26. Principal reinforcement in masonry shall be spaced 2 feet maximum on center in buildings using a moment resisting space frame.

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

D. **Diaphragms.** Floor and roof diaphragms shall be designed to resist the forces set forth in Table No. 23-J. Diaphragms supporting concrete or masonry walls shall have continuous ties between diaphragm chords to distribute, into the diaphragm, the anchorage forces specified in this Chapter. Added chords may be used to form sub-diaphragms to transmit the anchorage forces to the main cross ties. Diaphragm deformations shall be considered in the design of the supported walls. See Section 2312 (j) 3 A for special anchorage requirements of wood diaphragms.

3. **Special requirements.** A. **Wood diaphragms providing lateral support for concrete or masonry walls.** Where wood diaphragms are used to laterally support concrete or masonry walls the anchorage shall conform to Section 2310. In Zones No. 2, No. 3 and No. 4 anchorage shall not be accomplished by use of toe nails, or nails subjected to withdrawal; nor shall wood framing be used in cross grain bending or cross grain tension.

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

C. **Exterior elements.** Precast, nonbearing, nonshear wall panels or similar elements which are attached to or enclose the exterior, shall accommodate movements of the structure resulting from lateral forces or temperature changes. The concrete panels or other elements shall be supported by means of cast-in-place concrete or by mechanical fasteners in accordance with the following provisions.

Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind or $(3.0/K)$ times story drift caused by required seismic forces; or ¼ inch, whichever is greater.

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

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

(k) **Essential Facilities.** Essential facilities are those structures or buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public. Such facilities shall include but not be limited to:

1. Hospitals and other medical facilities having surgery or emergency treatment areas.
2. Fire and police stations.
3. Municipal government disaster operation and communication centers deemed to be vital in emergencies.

The design and detailing of equipment which must remain in place and be functional following a major earthquake shall be based upon the requirements of Section 2312 (g) and Table No. 23-J. In addition, their design and detailing shall consider effects induced by structure drifts of not less than $(2.0/K)$ times the story drift caused by required seismic forces nor less than the story drift caused by wind. Special consideration shall also be given to relative movements at separation joints.

(1) **Earthquake Recording Instrumentations.** For earthquake recording instrumentations see Appendix, Section 2312 (1).

[3]See Section 2306 for live load reductions. The rate of reduction r in Section 2306 Formula (6-1) shall be as indicated in the Table. The maximum reduction R shall not exceed the value indicated in the Table.

[4]As defined in Section 4506.

TABLE NO. 23-D—MAXIMUM ALLOWABLE DEFLECTION FOR STRUCTURAL MEMBERS[1]

TYPE OF MEMBER	MEMBER LOADED WITH LIVE LOAD ONLY (L.L.)	MEMBER LOADED WITH LIVE LOAD PLUS DEAD LOAD (L.L. + K D.L.)
Roof Member Supporting Plaster or Floor Member	$L/360$	$L/240$

[1]Sufficient slope or camber shall be provided for flat roofs in accordance with Section 2305 (f).

L.L. = Live load

D.L. = Dead load

K = Factor as determined by Table No. 23-E

L = Length of member in same units as deflection

TABLE NO. 23-E—VALUE OF "K"

WOOD		REINFORCED CONCRETE[2]	STEEL
Unseasoned	Seasoned[1]		
1.0	0.5	$[2 - 1.2\,(A'_s/A_s)] \geqq 0.6$	0

[1]Seasoned lumber is lumber having a moisture content of less than 16 percent at time of installation and used under dry conditions of use such as in covered structures.

[2]See also Section 2609.

A'_s = Area of compression reinforcement.

A_s = Area of nonprestressed tension reinforcement.

TABLE NO. 23-F—WIND PRESSURES FOR VARIOUS HEIGHT ZONES ABOVE GROUND[1]

HEIGHT ZONES (in feet)	WIND-PRESSURE-MAP AREAS (pounds per square foot)						
	20	25	30	35	40	45	50
Less than 30	15	20	25	25	30	35	40
30 to 49	20	25	30	35	40	45	50
50 to 99	25	30	40	45	50	55	60
100 to 499	30	40	45	55	60	70	75
500 to 1199	35	45	55	60	70	80	90
1200 and over	40	50	60	70	80	90	100

[1]See Figure No. 4. Wind pressure column in the table should be selected which is headed by a value corresponding to the minimum permissible, resultant wind pressure indicated for the particular locality.

The figures given are recommended as minimum. These requirements do not provide for tornadoes.

TABLE NO. 23-G—MULTIPLYING FACTORS FOR WIND PRESSURES—CHIMNEYS, TANKS, AND SOLID TOWERS

HORIZONTAL CROSS SECTION	FACTOR
Square or rectangular	1.00
Hexagonal or octagonal	0.80
Round or elliptical	0.60

TABLE NO. 23-H—SHAPE FACTORS FOR RADIO TOWERS AND TRUSSED TOWERS

TYPE OF EXPOSURE	FACTOR
1. Wind normal to one face of tower	
Four-cornered, flat or angular sections, steel or wood	2.20
Three-cornered, flat or angular sections, steel or wood	2.00
2. Wind on corner, four-cornered tower, flat or angular sections	2.40
3. Wind parallel to one face of three-cornered tower, flat or angular sections	1.50
4. Factors for towers with cylindrical elements are approximately two-thirds of those for similar towers with flat or angular sections	
5. Wind on individual members	
Cylindrical members	
Two inches or less in diameter	1.00
Over two inches in diameter	0.80
Flat or angular sections	1.30

TABLE NO. 23-I—HORIZONTAL FORCE FACTOR "K" FOR BUILDINGS OR OTHER STRUCTURES[1]

TYPE OR ARRANGEMENT OF RESISTING ELEMENTS	VALUE[2] OF K
1. All building framing systems except as hereinafter classified	1.00
2. Buildings with a box system as specified in Section 2312 (b)	1.33
3. Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls or braced frames using the following design criteria: a. The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames b. The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces c. The ductile moment resisting space frame shall have the capacity to resist not less than 25 percent of the required lateral force	0.80
4. Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force	0.67
5. Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building	2.5[3]
6. Structures other than buildings and other than those set forth in Table No. 23-J	2.00

[1]Where wind load as specified in Section 2311 would produce higher stresses, this load shall be used in lieu of the loads resulting from earthquake forces.

[2]See Figure Nos. 1, 2 and 3 this chapter and definition of "Z" as specified in Section 2312 (c).

[3]The minimum value of "KC" shall be 0.12 and the maximum value of "KC" need not exceed 0.25.

The tower shall be designed for an accidental torsion of five percent as specified in Section 2312 (e) 5. Elevated tanks which are supported by buildings or do not conform to type or arrangement of supporting elements as described above shall be designed in accordance with Section 2312 (g) using "C_p" = .2.

TABLE NO. 23-J—HORIZONTAL FORCE FACTOR "C_p" FOR ELEMENTS OF STRUCTURES

PART OR PORTION OF BUILDINGS	DIRECTION OF FORCE	VALUE OF C_p
1. Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions. Masonry or concrete fences	Normal to flat surface	0.20[1]
2. Cantilever parapet	Normal to flat surface	1.00
3. Exterior and interior ornamentations and appendages.	Any direction	1.00
4. When connected to, part of, or housed within a building: a. Towers, tanks, towers and tanks plus contents, chimneys, smokestacks and penthouse	Any direction	0.20[2]
b. Storage racks with the upper storage level at more than 8 feet in height plus contents		0.20[2,3]
c. Equipment or machinery not required for life safety systems or for continued operations of essential facilities		0.20[2,4]
d. Equipment or machinery required for life safety systems or for continued operation of essential facilities		0.50[4,5]
5. When resting on the ground, tank plus effective mass of its contents.	Any direction	0.12
6. Suspended ceiling framing systems (Applies to Seismic Zones Nos. 2, 3 and 4 only)	Any direction	0.20[6]
7. Floors and roofs acting as diaphragms	Any direction	0.12[7]
8. Connections for exterior panels or for elements complying with Section 2312 (j) 3C.	Any direction	2.00
9. Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any direction	0.30[8]

[1]See also Section 2309 (b) for minimum load on deflection criteria for interior partitions.

[2]When located in the upper portion of any building where the h_n/D ratio is five-to-one or greater the value shall be increased by 50 percent.

[3]W_p for storage racks shall be the weight of the racks plus contents. The value of C_p for racks over two storage support levels in height shall be 0.16 for the levels below the top two levels. In lieu of the tabulated values steel storage racks may be designed in accordance with U.B.C. Standard No. 27-11.

(Continued)

Where a number of storage rack units are interconnected so that there are a minimum of four vertical elements in each direction on each column line designed to resist horizontal forces, the design coefficients may be as for a building with K values from Table No. 23-I, $CS = 0.20$ for use in the formula $V = ZIKCSW$ and W equal to the total dead load plus 50 percent of the rack rated capacity. Where the design and rack configurations are in accordance with this paragraph the design provisions in U.B.C. Standard No. 27-11 do not apply.

[4]For flexible and flexibly mounted equipment and machinery, the appropriate values of C_p shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but shall not be less than the listed values. The design of the equipment and machinery and their anchorage is an integral part of the design and specification of such equipment and machinery.

[5]For Essential Facilities and life safety systems, the design and detailing of equipment which must remain in place and be functional following a major earthquake shall consider drifts in accordance with Section 2312 (k). The product of IS need not exceed 1.5.

[6]Ceiling weight shall include all light fixtures and other equipment which are laterally supported by the ceiling. For purposes of determining the lateral force, a ceiling weight of not less than 4 pounds per square foot shall be used.

[7]Floors and roofs acting as diaphragms shall be designed for a minimum force resulting from a C_p of 0.12 applied to w_x unless a greater force results from the distribution of lateral forces in accordance with Section 2312 (e).

[8]The W_p shall include 25 percent of the floor live load in storage and warehouse occupancies.

TABLE NO. 23-K
VALUES FOR OCCUPANCY IMPORTANCE FACTOR I

TYPE OF OCCUPANCY	I
Essential Facilities[1]	1.5
Any building where the primary occupancy is for assembly use for more than 300 persons (in one room)	1.25
All others	1.0

[1]See Section 2312 (k) for definition and additional requirements for essential facilities.

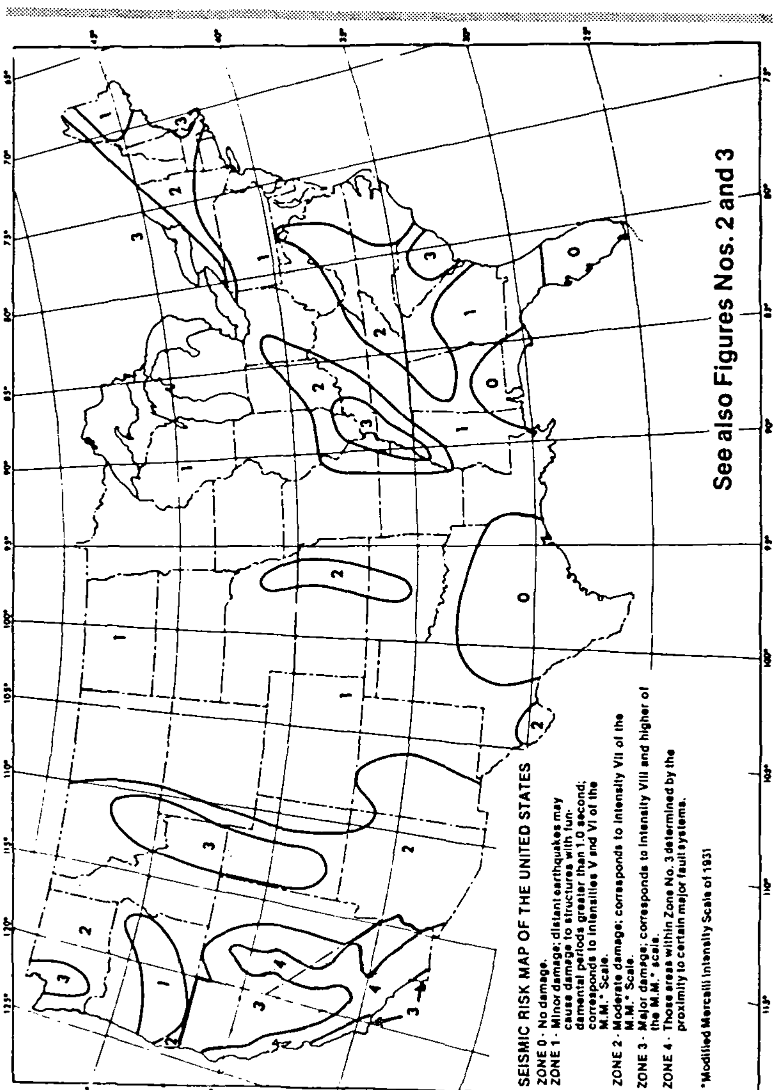

FIGURE NO. 1—SEISMIC ZONE MAP OF THE UNITED STATES
For areas outside of the United States see Appendix Chapter 23

from the

UNIFORM BUILDING CODE

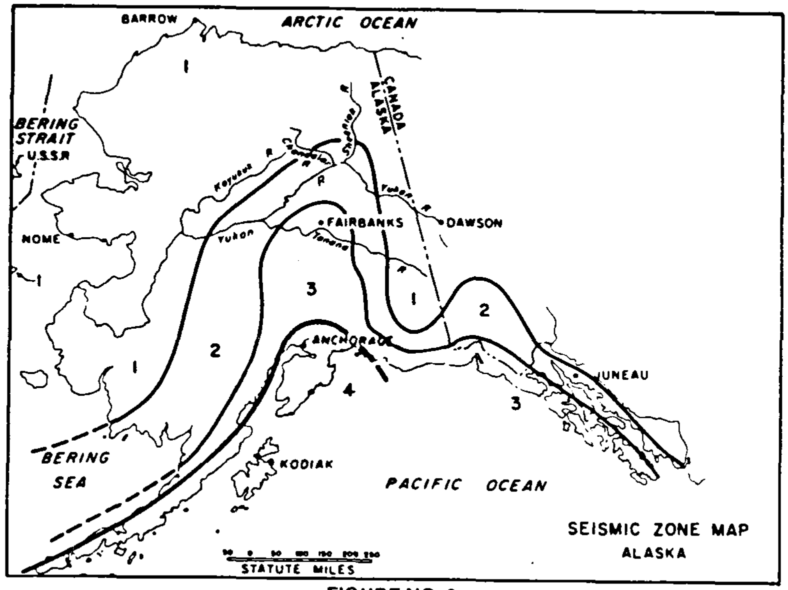

FIGURE NO. 2

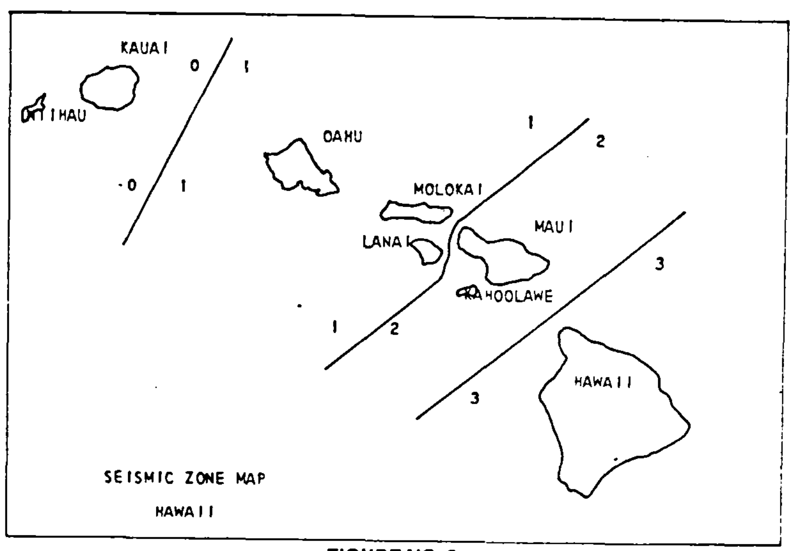

FIGURE NO. 3

from the

UNIFORM BUILDING CODE

7.7 Snow Load Design

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SNOW LOADS

Basic Snow Loads

Snow loads on roofs vary widely throughout the United States. Factors affecting snow load accumulation on roofs include climatic variables of elevation, latitude, wind frequency and duration of snowfall, roof geometry, and site exposure. In addition, snowfall varies from year to year, and either a mean recurrence interval must be established for design purposes or design should be based on the maximum recorded snow load for which data is available. Snow loads should be as stipulated by the governing building code, but in the absence of such a code, snow loading used for design should be based on local investigation, or by the use of accepted snow load maps.

Before discussing roof loads, it is necessary to consider ground snow loads since these form the basis for determining roof loads. Several researchers and agencies have measured ground snow load distribution and plotted appropriate isogram maps depicting these loads. Figure 3.1 presents maximum snow loads on the ground based on records of the U.S. Weather Bureau. Recent data reported by H. C. S. Thom of the Environmental Data Service, U.S. Department of Commerce, in "Distribution of Maximum Annual Water Equivalent of Snow on the Ground" and published in the *Monthly Weather Review*, Vol. 94, No. 4, 1966, provides snow load maps based on 25-, 50-, and 100-year mean recurrence intervals. In general, the 50-year mean recurrence data as prepared by Thom corresponds generally to the values listed in Figure 3.1 with slightly higher values recorded for the Northeastern states and somewhat lower values plotted for the Western states.

For the Western states, Figure 3.1 indicates that the snow loads for these regions should be established based on local experience. Actual snow pack in these regions of over 700 psf has been recorded and many of the inhabited regions have snow loads of 100, 200, or 300 psf.

L. W. Neubauer, in his paper "Snow Loads on Roofs in Mountain Regions," published in *Transactions*, ASAE, Vol. 12, No. 3, 1969, proposes the

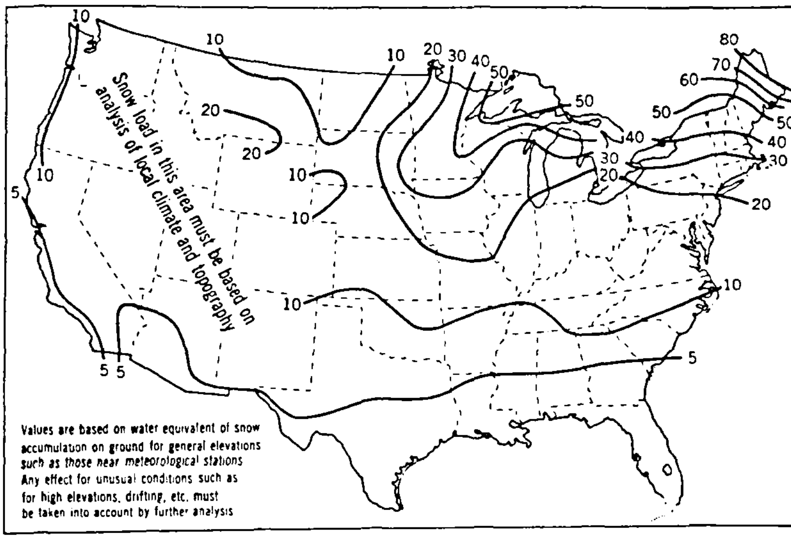


Figure 3.1 SNOW LOAD IN PSF ON THE GROUND, 50-YR MEAN RECURRENCE INTERVAL. Source: U.S. Weather Bureau, Washington, D.C.

use of the following equation to estimate ground snow loads in the Western mountain regions, based on a number of field studies conducted:

$$SL = 2.3\left(El. + \frac{L}{4} - 8.5\right)^2$$

where SL = ground snow load, psf
$El.$ = elevation, ft/1,000
L = latitude, degrees

Although this equation is strictly empirical, it should be helpful in estimating snow loads at various elevations and latitudes when satisfactory snow-load maps or data are not available.

Roof Snow Loads

Based on a determination of the ground snow load as specified by the governing building code, through the use of maximum snow load maps such as given in Fig. 3.1, by the use of snow load maps based on a specific mean recurrence interval as developed by Thom, or by the use of an equation such as the one suggested by Neubauer, it is next necessary to determine the actual snow loads to be expected on the roof surface. As indicated, the roof snow load is a function of the geometry of the roof and the exposure to wind forces.

These factors can be accounted for in design by applying appropriate snow load coefficients to the basic ground snow loads. Specific snow load coefficients have been developed to relate roof snow load to ground snow load based on comprehensive surveys of actual conditions. This approach has been adopted by Canada and is included in the National Building Code of Canada.

For the design of both ordinary and multiple series roofs, either flat, pitched, or curved, a basic snow load coefficient of 0.8 should be used to convert ground snow load to a roof snow load. This value should then be increased or decreased if necessary because of specific roof geometry conditions, such as decreasing for roof slopes exceeding 20° and increasing for roofs having valleys formed by multiple series or other similar geometry conditions. When roof slopes exceed 20°, the design load in PSF may be decreased by (SL/40-½) for each 1° of slope over 20°. Specific coefficients for these roof configurations are given in the *American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures* by the American National Standards Institute.

For roofs exposed to winds of sufficient intensity to blow snow off, the basic snow load coefficient can be reduced to 0.6. This coefficient is only applicable if (*a*) the roof is not shielded from the wind on any side or is not likely to become shielded by obstructions higher than the roof within a distance of 10*h* from the building ("*h*" is the height of the obstruction above the roof level), and (*b*) the roof does not have any projections, such as parapet walls, which may prevent the snow from being blown off by the wind.

Since unbalanced loading can occur as the result of drifting, sliding, melting and refreezing, or physical removal of snow, structural roof members should be designed to resist the full snow load as defined above distributed over the entire roof area, the full snow load distributed on any one portion of the area, and dead load only on the remainder of the area, depending on which load produces the greatest stress on the member considered. With respect to duration of load, snow load duration is the cumulative time during which the maximum design load is on the structure over its entire life. A *2-month* duration is generally recognized as the proper design level for snow loads. Although some snow remains on roofs for periods exceeding 2 months in a single year, such snow loads seldom approach the design load.

Although the analysis of roofs for snow loading is complex due to the many variables involved, recent technical data developed as discussed in the preceding paragraphs have provided the engineer with sufficient information to make a realistic analysis.

8.0 APPENDICES

Appendix A

Alternate Systems and Materials

Space Frames

Structural space frames are based on the use of a few simple repetitive units. These units are fabricated with square tube, angle, pipe or other shapes in steel, aluminum or stainless steel. Sizes, depths and space frame dimensions and support locations are flexible. The basic unit (or module) can be expanded into larger units by simply connecting repetitive units. This repetitive technique allows the designer to construct virtually any size structure he chooses.

Space frames are usually fabricated and erected by various combinations of shop fabrication with field assembly, depending on frame type (tube, pipe, angle, flange) design requirements and local conditions. Numbers of members, joints, and bolts per joint are kept to a minimum, reducing the number and size of field connections, thus reducing field erection time and costs.

Some advantages to consider are:

- Standard units available from stock, quick delivery
- Easy transport, handling and stacking
- Fast, economical fabrication and erection

Publications covering standard details and structural performance data for space frames are readily available from the various manufacturers. The general comprehensive nature of these publications allow the designer ease in calculating for required loads.

Stock Framing Systems

There are two basic stock framing system types available; one employing channels as the main structural member and the other employing pipe.

The "channel type" consists of various sizes of channel (some perforated) utilizing a special spring-loaded nut. This special nut can be inserted anywhere along the continuous channel slot, held in position by the spring, allowing for the securing with bolts, appropriate fittings. The leading manufacturer of this type of system offers over a dozen basic channels, dozens of combination channels and over a thousand different types of fittings, making it possible to construct an infinite variety of forms and structures.

Care should be taken in designing with this type of system to remain cost-competitive with traditional materials.

The designer/engineer might consider the use of components of this system with other techniques. Many of the fittings offered by this system can be readily used with other forms.

The second type in this category is the "pipe and slip-on pipe fitting" system. This system is designed to be used with standard iron 1.3 to 5.1 cm (1/2 to 2 in.) pipe, available from any plumbing or mill supply house. The fittings are usually made of malleable iron with from one to five case hardened set screws to each fitting depending upon the number of pipes to be joined at one point. Several dozen types of fittings are offered giving unlimited scope to the designer/engineer, making possible the fabrication of the unusual type of structure as well as the simple structure. However, most structures can be designed from just two or three types of fittings.

Stock Perforated and Slotted Shapes

Stock perforated and slotted shapes are a versatile framing material usually made from 2.7 to 1.5 mm thick (12 to 16 gage) cold-formed steel. Among the shapes available are even and uneven leg angles and channels, in 1.2 to 6.1 meter (4 to 20 ft) lengths. Slots and holes, either singularly or in combination, are located at fractional intervals allowing a broad range of flexibility in design. Individual members are joined together by bolts and nuts. Angular fittings can and are often used with this form.

Plastic Shapes

Pultrusions: Pultruded shapes are produced by pulling kesin (polyester, epoxy, etc.) impregnated fibers (glass, carbon, aramid) through a steel die, which determines the shape and controls the resin content. Almost any constant cross section can be formed continuously and by nature of the machine process, the finished product is uniform.

Pultruded elements have high strength-to-weight ratio and are virtually unbreakable in most applications. They're approximately two-thirds the weight of aluminum, are unaffected by electrolytic corrosion, withstand chemical corrosion from a wide variety of corrosive liquids and gases, provide electrical insulation and are excellent thermal insulators.

The following structural shapes and profiles are standard stock items: equal leg angle, channels, I-beams (W shapes), wide flange beams, round tube, square tube, rectangular tubing, round rod, flat sheet and square bar.

The pultruded shapes can be fabricated by most conventional methods such as sawing, drilling, routing, tapping, or machining.

The individual members or elements can be joined by bonding and bolting, or bonding and riveting.

Fabrication and engineering manuals available from the manufacturers offer the designer/engineer a comprehensive profile of this new engineering material.

Plastic Pipe and Fittings

Another possible, innovative structural system that the designer/engineer might consider is one employing plastic pipe and plastic fittings. This form of system is particularly suited to small structures.

Plastic pipe is made from PVC, CPVC, PE, PP, and PVF, ranging in size from 0.64 to 10.2 cm (1/4 to 4 in.) in diameter in approximately 6 meter (20 ft) lengths.

Fittings available are: 90° elbows, 45° elbows, tees, reducing tees, couplings, reducing couplings, flanges, caps, reducing flanges, reducing bushings, sweeps and long radius bends.

The members and elements of this system can be either threaded or cemented together. Flanges allow for bolting. Fabrication is easy and assembly in the field is accomplished with common tools.

Module fastening to this form of structure is by "U" or "J" bolts or by drilling through tube. A submember channel or angle might serve as interface between structure and cell module.

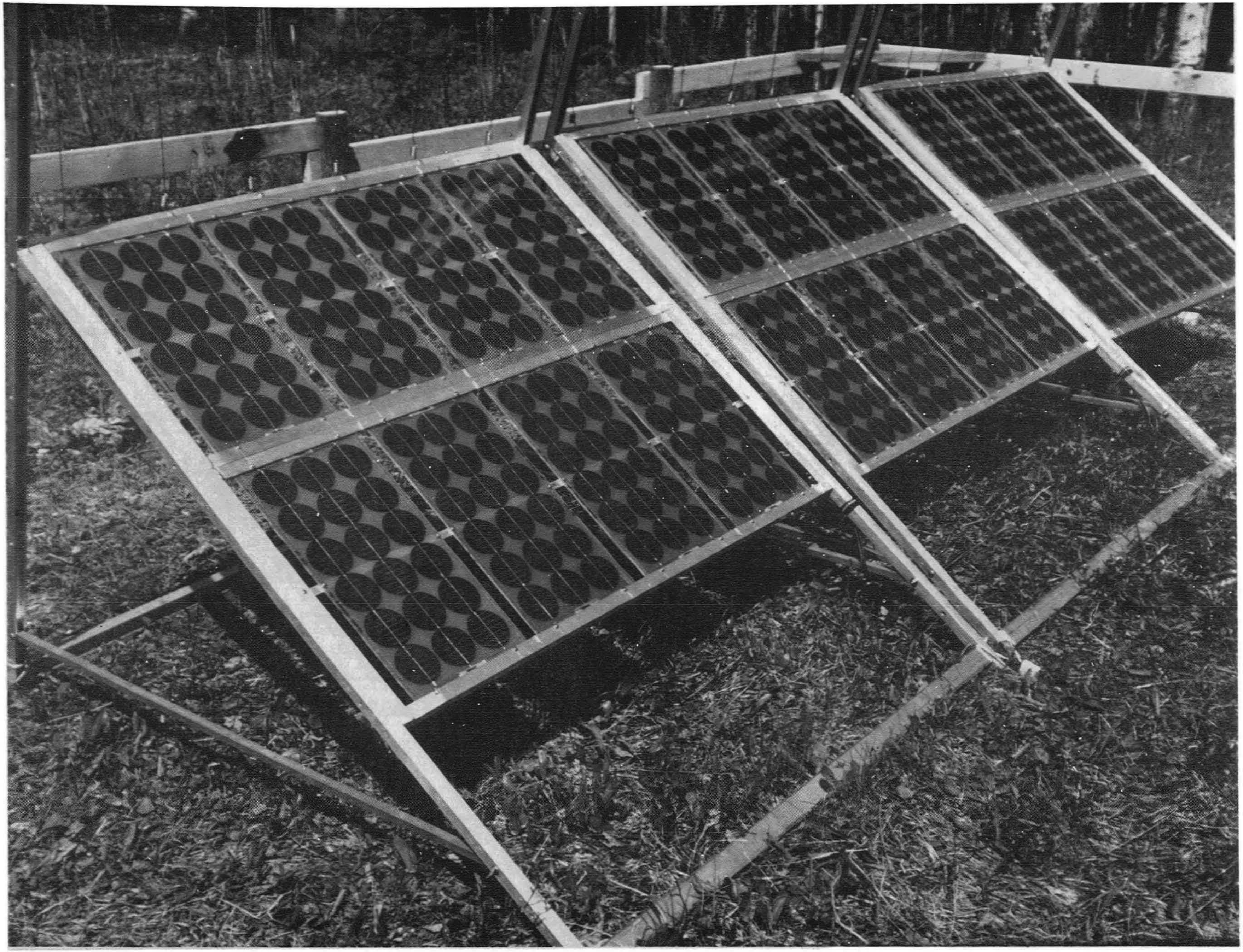
Appendix B

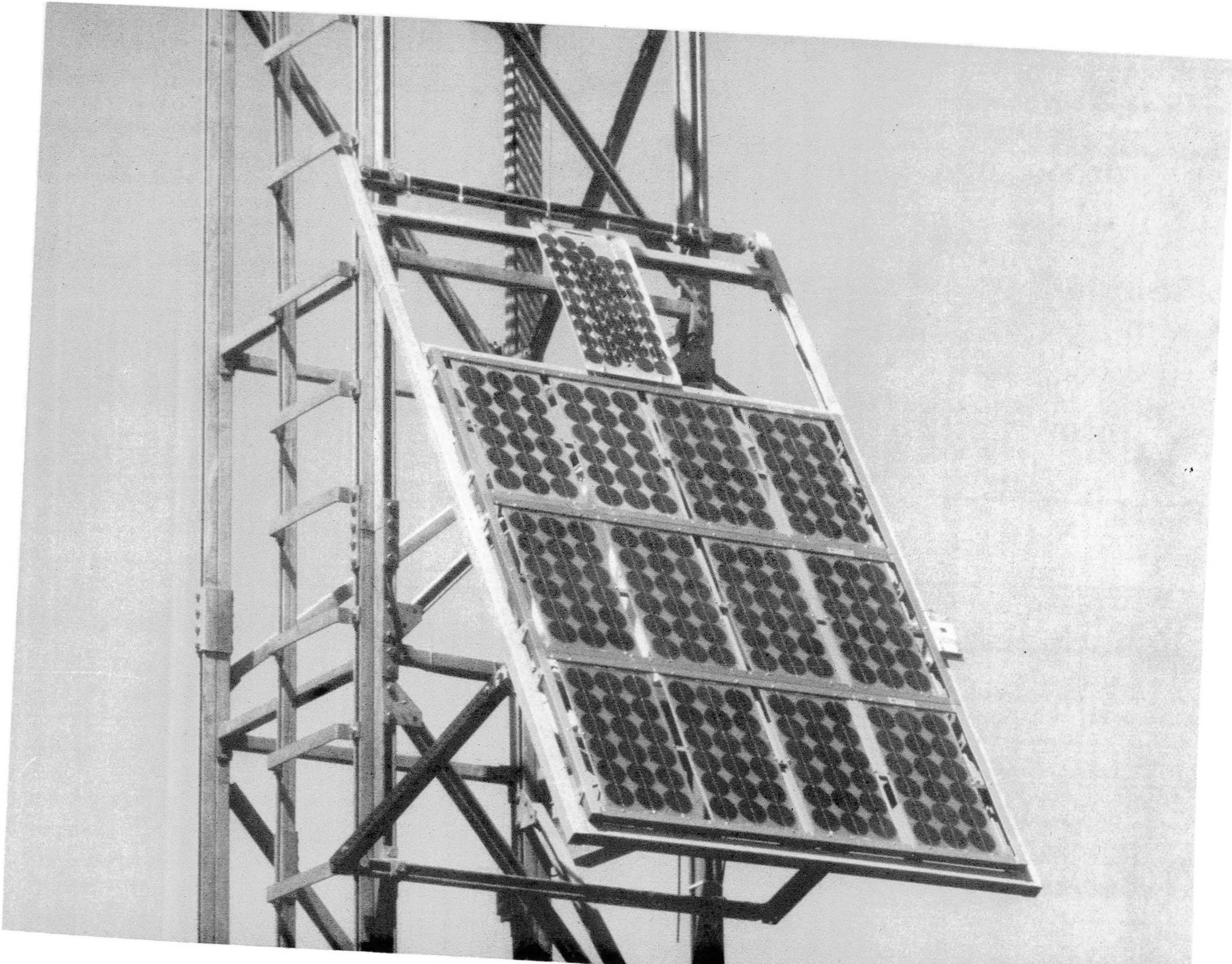
Photographs of Typical Small Photovoltaic Array Installations

This appendix consists of a series of photographs showing several small photovoltaic array installations fielded by the NASA Lewis Research Center for the Department of Energy and the Agency for International Development over the past several years.

While most of the arrays indicated in the photographs presented herein are different from those covered in the handbook, the photographs serve to show samplings of some of the different types of solar cell modules available. Further, the photographs illustrate the basic mounting techniques and structures that have been used in past experimental installations. It should be noted, however, that many of the structures depicted in the photographs were not very cost efficient. It should also be noted that the primary objective of these early experiments was to demonstrate the viability of photovoltaic systems to meet electrical needs in low power system applications, not cost efficiency.

NASA
C-76-2566





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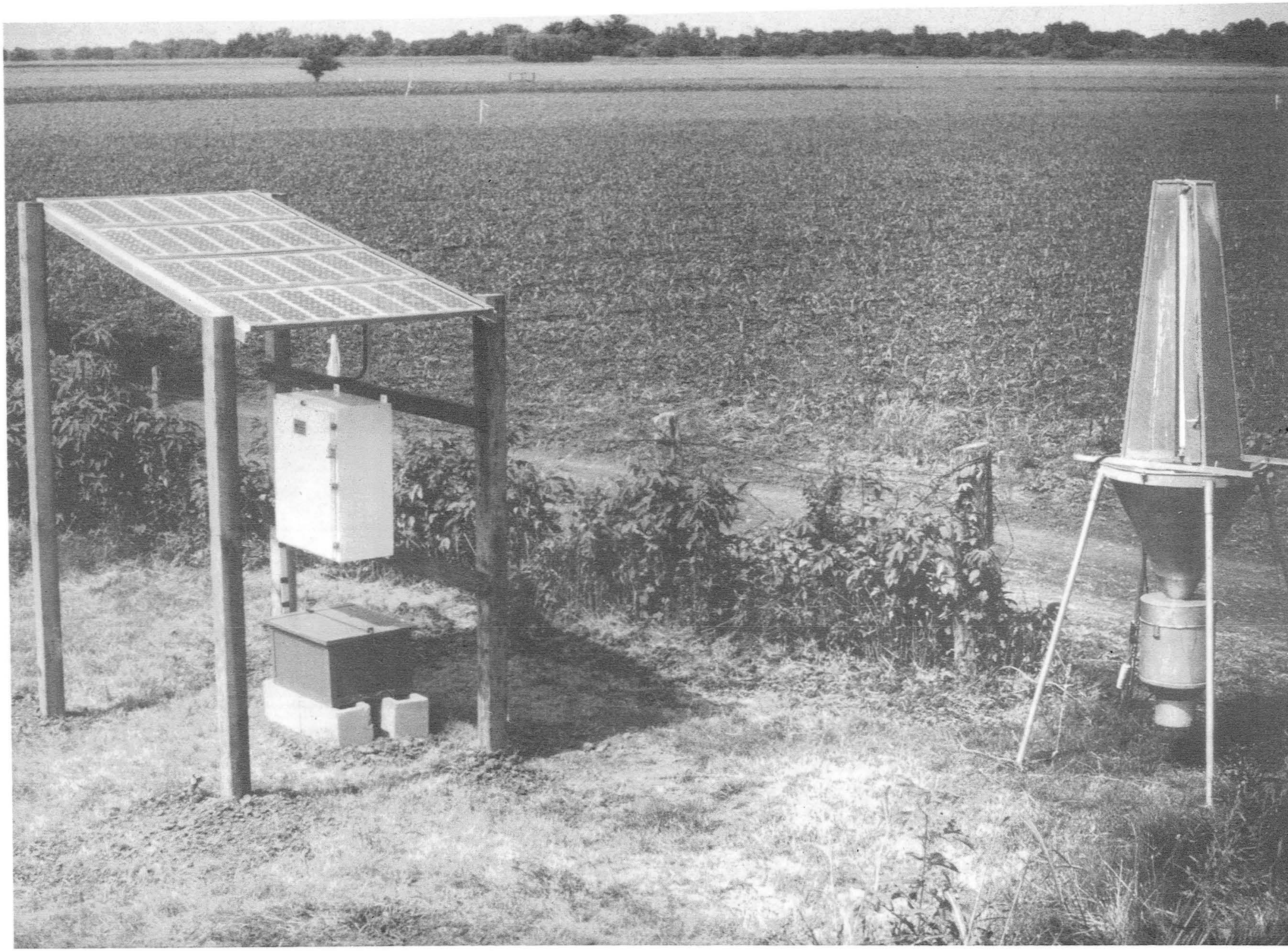

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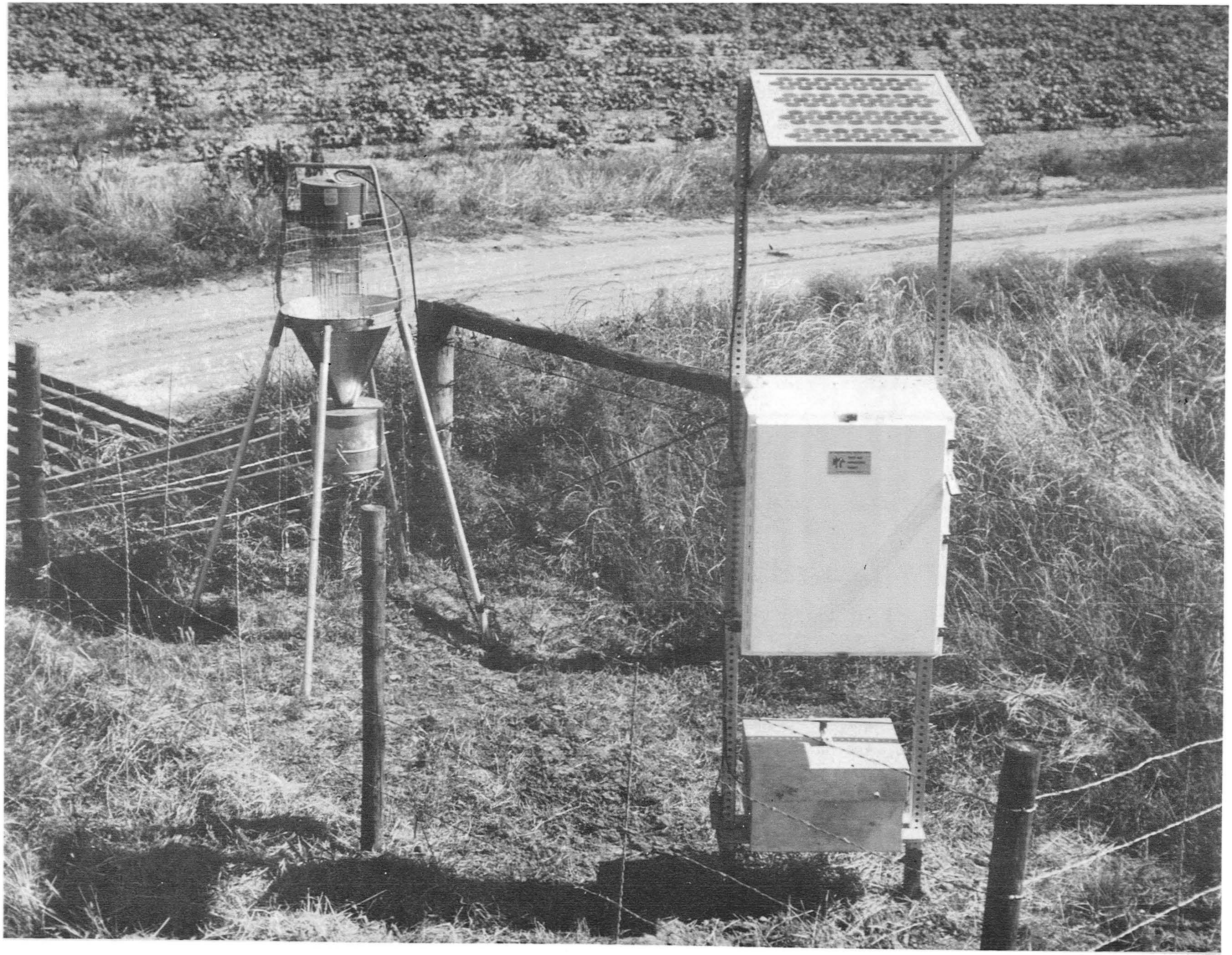

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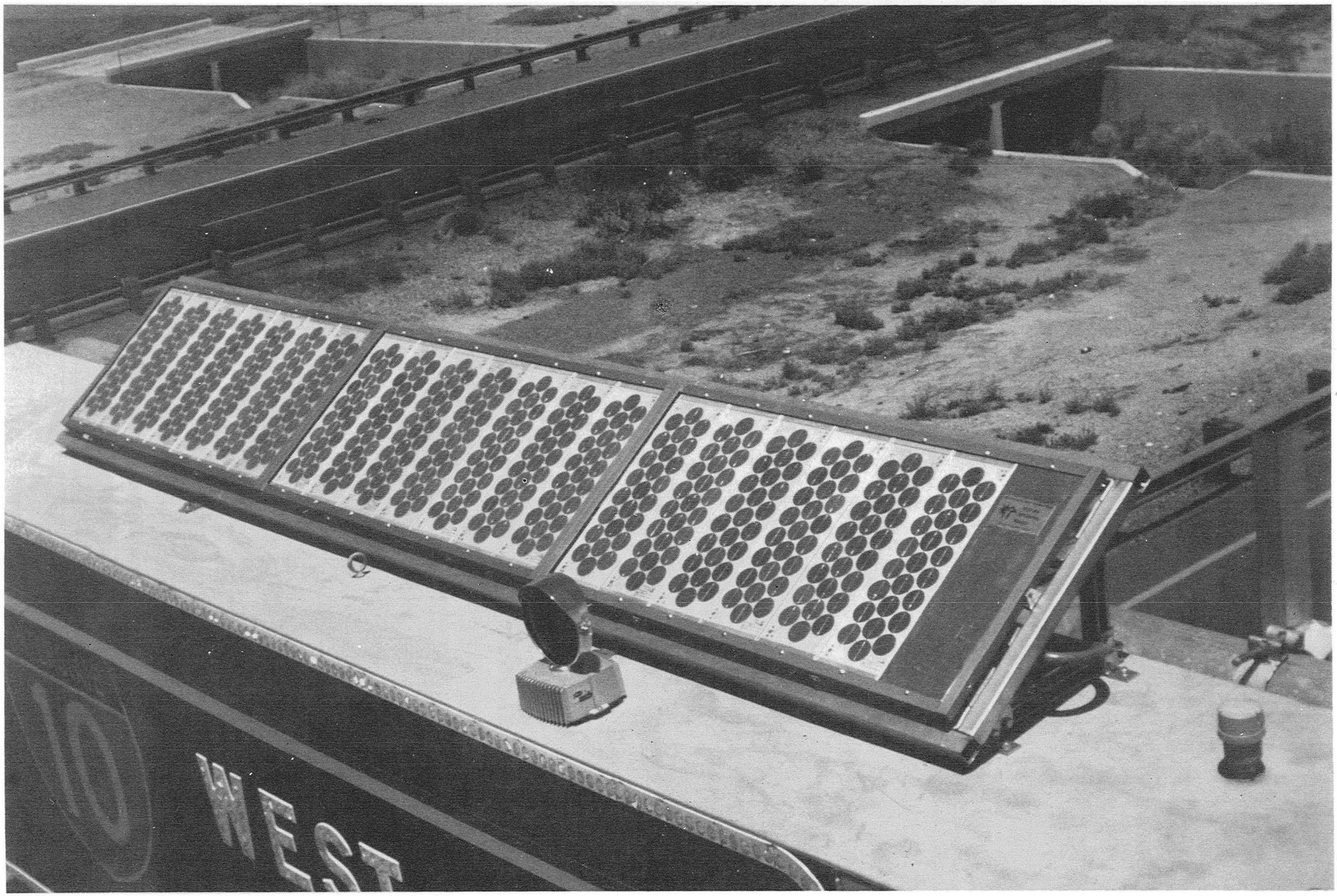

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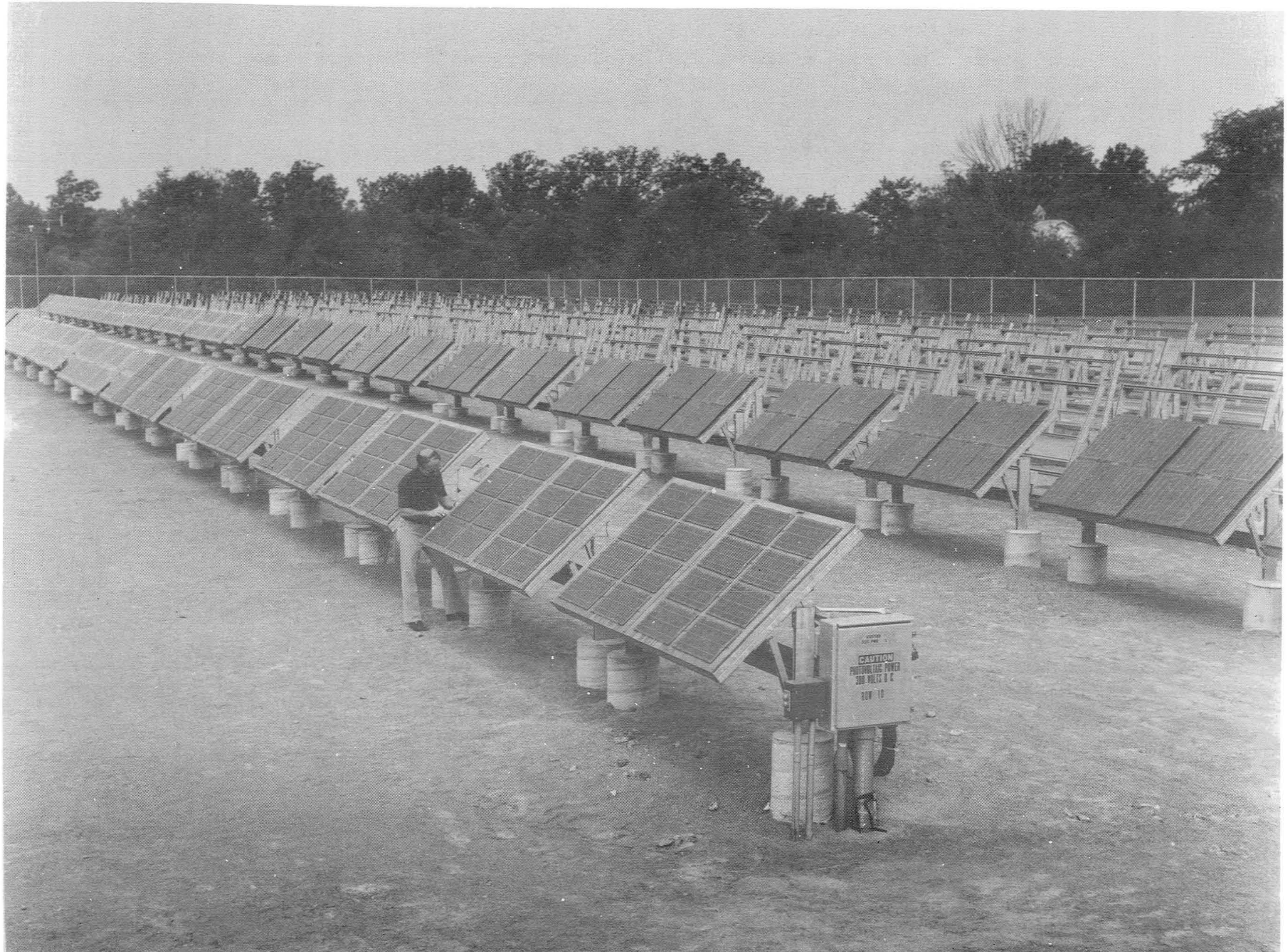

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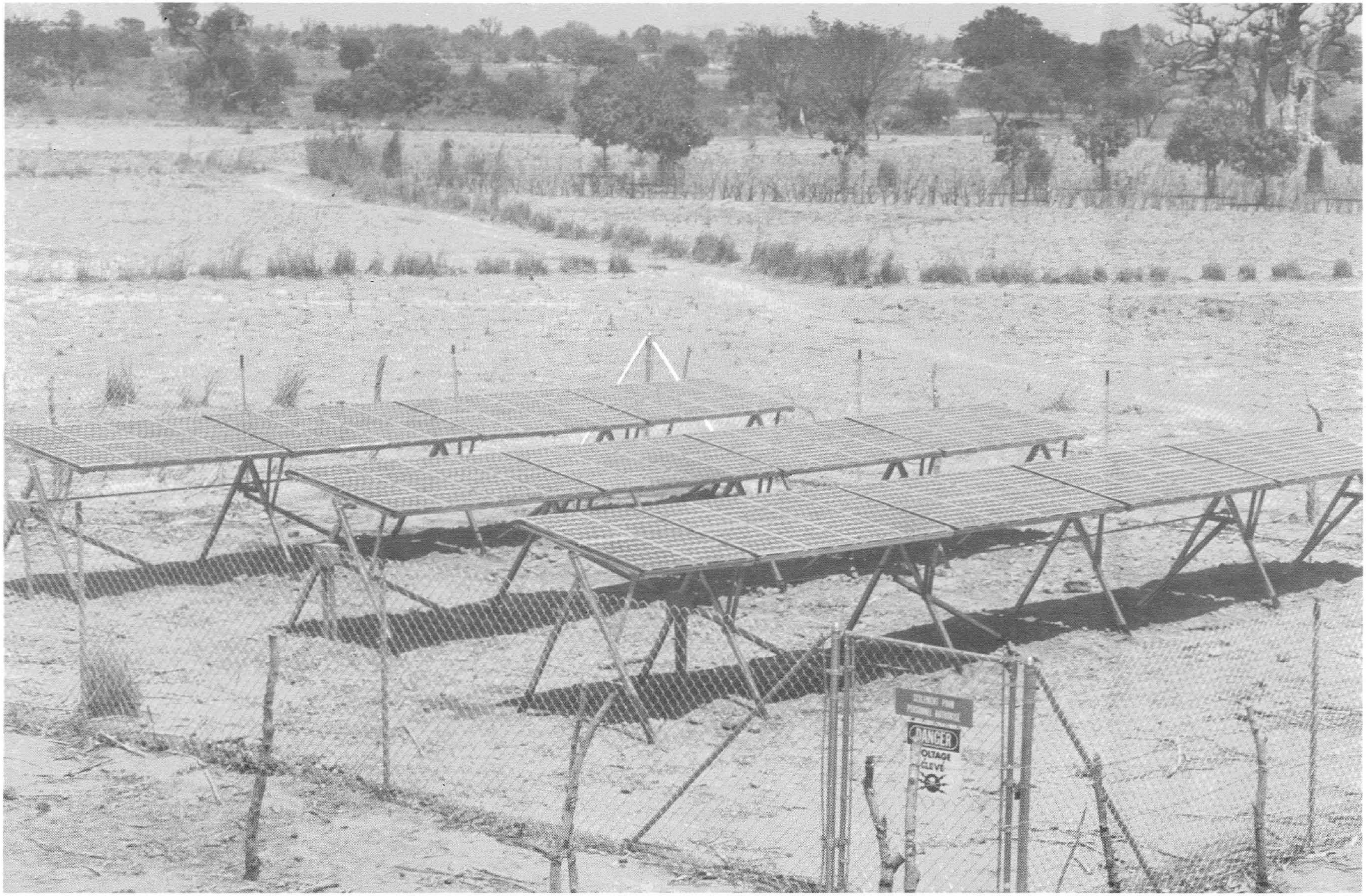

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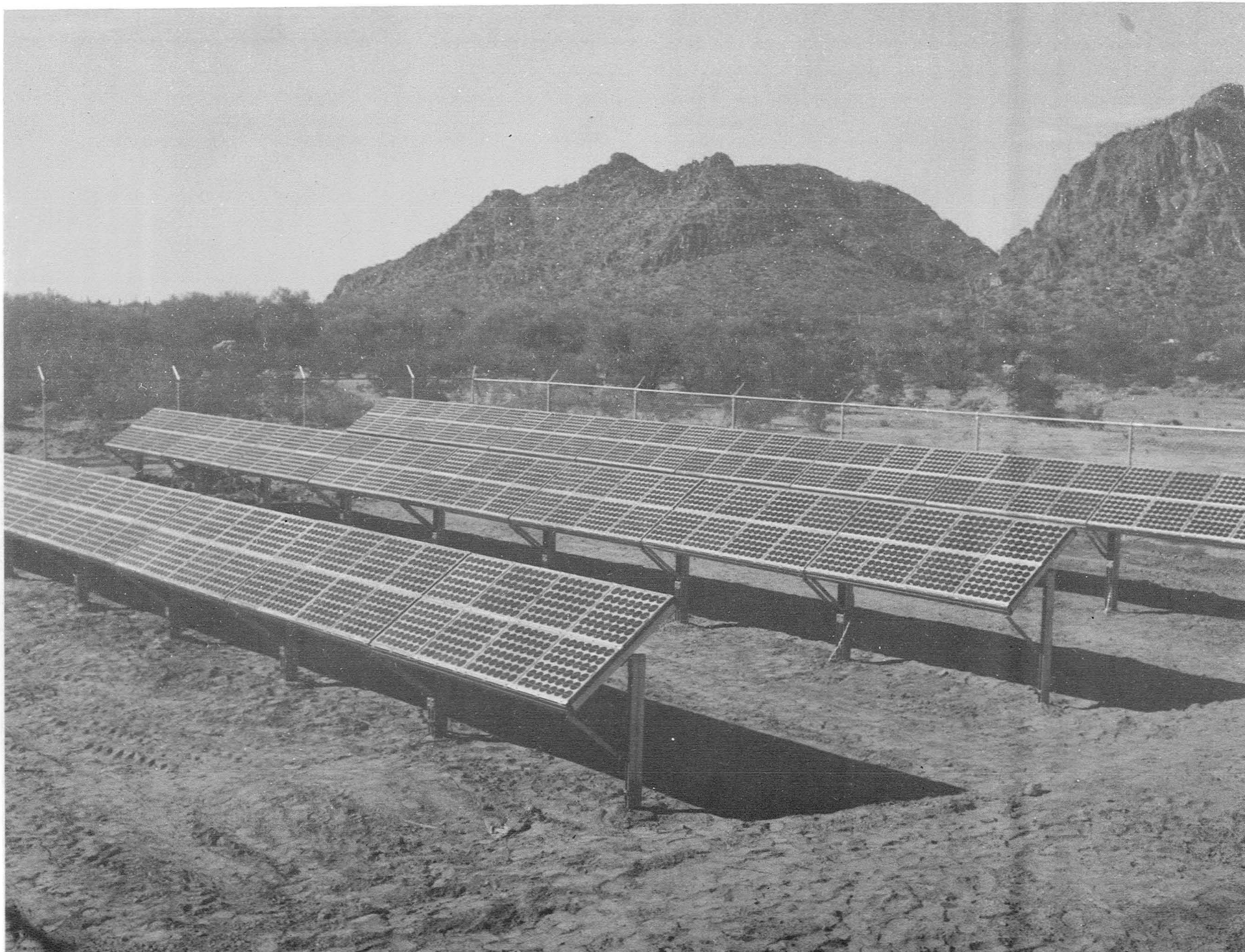

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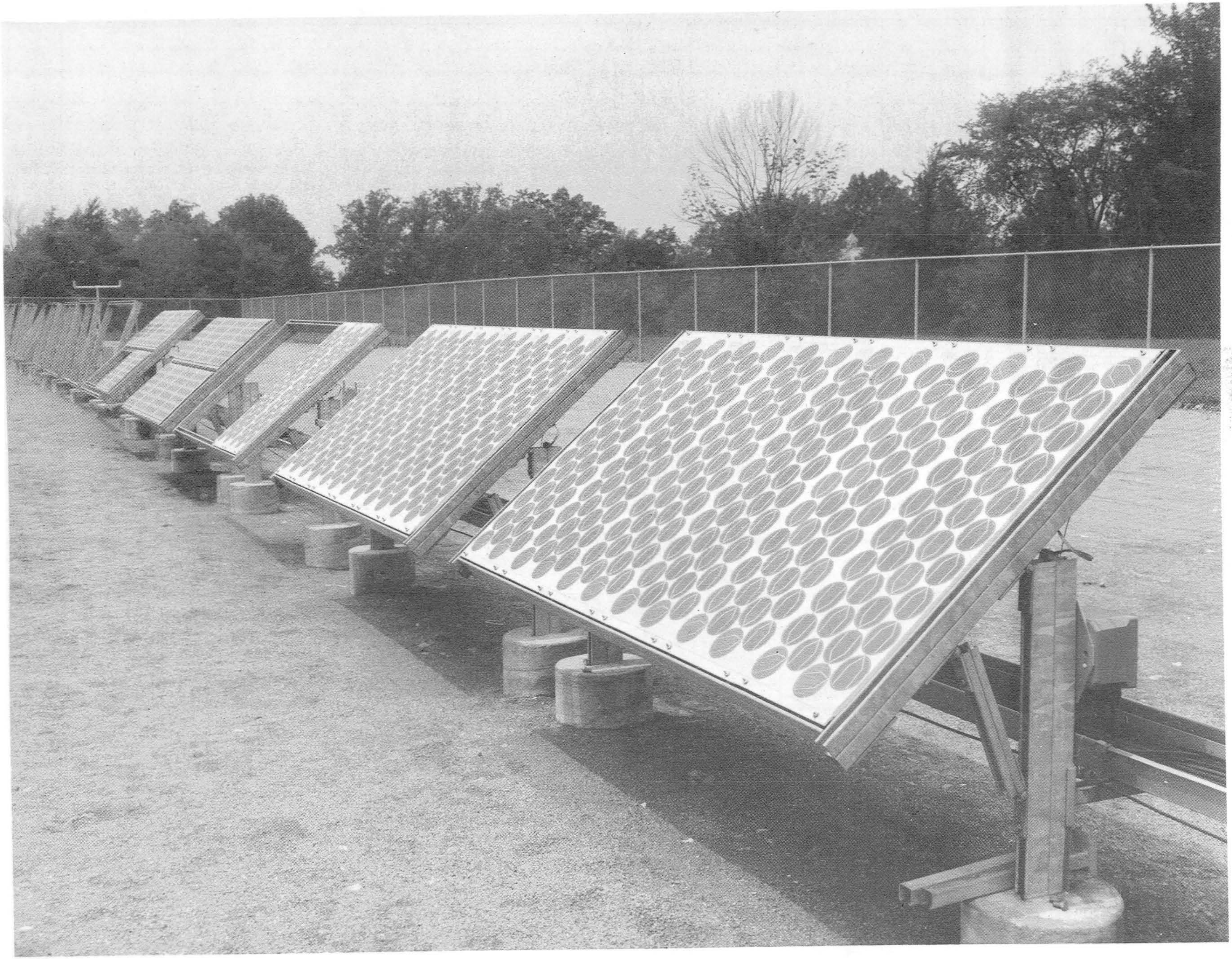

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16. Abstract

This handbook will permit the user to design a low-cost structure for a variety of photovoltaic system applications under 10 kW. Any presently commercially available photovoltaic modules may be used. Design alternatives are provided for different generic structure types, structural materials, and electric interfaces. The use of a hand-held calculator is sufficient to perform the necessary calculations for the array designs.

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