Prestressed Concrete Poles: State-of-the-Art

Thomas E. Rodgers, Jr.
Director Structural Engineering
VEPCO, Virginia Electric and
Power Company
Richmond, Virginia

For many years throughout the world, poles made of wood, steel, and concrete have been used to support power transmission, telephone and telegraph lines, street lighting, overhead power lines for railroads, and antenna masts. Concrete poles were first used over 60 years ago and were then made of normal reinforced concrete. As technology improved, production and use of concrete poles gradually increased.

Prestressed concrete poles should not be regarded as novel or new. They were first designed and constructed by the French prestressing pioneer Eugéne Freyssinet in Algeria in the middle 1930s.

It was also Freyssinet who, many years ago, conducted his classic demonstration in which a normal reinforced concrete pole and a prestressed pole, designed to the same ultimate load, were placed in a special testing apparatus and subjected, not only to repetition of a load equal to, but also to alternation of a load amounting to 50 percent of the designed ultimate load. The normal reinforced concrete pole failed at a few thousand repetitions, but the prestressed pole was still going strong at 500,000 cycles. Since this is the type of loading that poles are expected to carry in the field, this test was of significant interest to pole users.

The greatest hazard associated with normal reinforced concrete poles is corrosion of steel. This leads to spalling of the concrete and, ultimately, failure of the pole. The corrosion may be caused by insufficient cover, substandard concrete, or excessive tensile forces, hence cracks, in handling or service, owing to poor design or to misuse. Any of these conditions can cause penetration of

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water to the steel. The process might take several years, but once corrosion is started, failure becomes inevitable.

The prestressed concrete pole offers the following advantages: First, the concrete used is of a quality sufficient to resist penetration of water, otherwise the pole could conceivably fail during the prestressing operation. Secondly, in a prestressed pole, the concrete is usually in compression, and cracking is not possible except under abnormal conditions of handling or service. These characteristics give the prestressed pole greater advantages over the normal reinforced pole and are the reasons for the development and use of prestressed concrete poles.

Today, prestressed concrete poles are widely used throughout Eastern and Western Europe. They are extensively utilized in Japan and to varying degrees in many other countries around the world. Available data indicate that the Soviet Union has produced and used the most poles, whereas the United States and Canada have only recently initiated their utilization.

Synopsis

Presents an historical and state-of-the-art review of the application, design, manufacture, testing, handling, transportation, and erection of prestressed concrete poles throughout the world. Particular emphasis is given to centrifugally spun precast concrete poles.

Design criteria (especially load selection) are discussed in detail together with the relevant provisions from the various codes of practice. It is concluded that prestressed concrete poles will play an increasingly dominant role in the future.

SCOPE OF APPLICATION

Prestressed concrete poles have found increasingly wide acceptance due to their viability as a factory produced unit. It is reported that East and West Germany, Poland, and Czechoslovakia each manufacture 50,000 to 100,000 poles a year. In the Soviet Union the annual production amounts to several hundred thousand concrete poles. France reportedly uses more than 400,000 concrete poles every year from its 14 factories producing this unit. In Japan, Nippon Concrete Industries, the leading manufacturer of spun concrete poles, produces more than 5400 tons (4900 t) of prestressed concrete poles a month in their eight factories. In 1979 a group of American engineers visited a concrete pole plant belonging to the Shanghai Electricity Bureau in China, which started spinning regular reinforced concrete poles in 1958 and prestressed poles in 1968 (Figs. 1 and 2).

In East Germany prestressed concrete poles find application in many fields. In the following industries, of the total number of poles, the percentages for prestressed concrete are: for electric power transmission lines, 60 percent; for overhead wires on railways, 30 percent; for lighting systems, 70 percent; and for other uses, 20 percent.

In Norway not many prestressed poles have been produced for electric power transmission because the terrain makes their transportation too difficult and expensive. However, the use of prestressed poles facilitated the electrification of railroads in mountainous areas.

In Poland and other Eastern European countries various types of prestressed concrete poles are used for telephone and electric power transmission lines.

In Western Europe spun prestressed concrete poles are widely used for electric power transmission, railroads,

Fig. 1. Spun concrete poles (Shanghai Electricity Bureau).

Fig. 2. Closeup of spun concrete pole (Shanghai Electricity Bureau).

lighting masts and antennas, and communication masts. The British Railways have extensively used prestressed poles for carrying overhead transmission. Figs. 3-10 show these applications.

In the Soviet Union reinforced concrete poles were first used for electric power transmission lines in 1933. During World War II, reinforced concrete poles began to be used more widely. Many poles were needed on the electric grid as replacements for those destroyed and for expanding the system. The growth of the precast concrete industry facilitated the manufacture of these poles in various types and sizes.

The electrification of the railroad system is of great importance to the Soviet Union. Every year about 1250 miles (2000 km) of track are electrified, and the use of prestressed concrete poles to carry the overhead cables have been a major feature of the work. The production of concrete poles for overhead cables on railroads reached

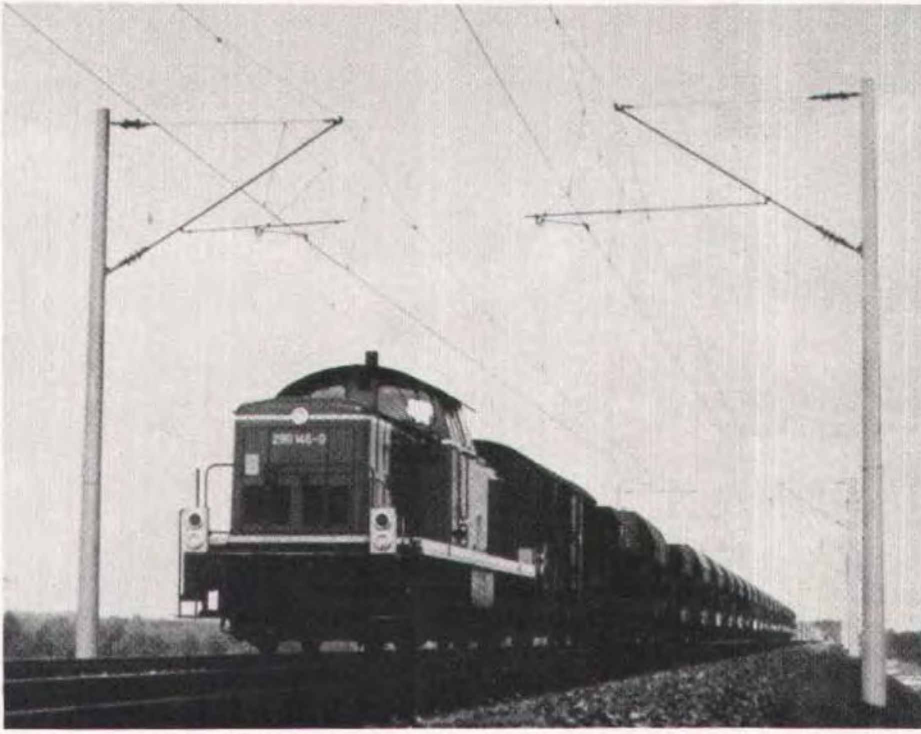


Fig. 3. Railroad electrification structures (Western Europe).

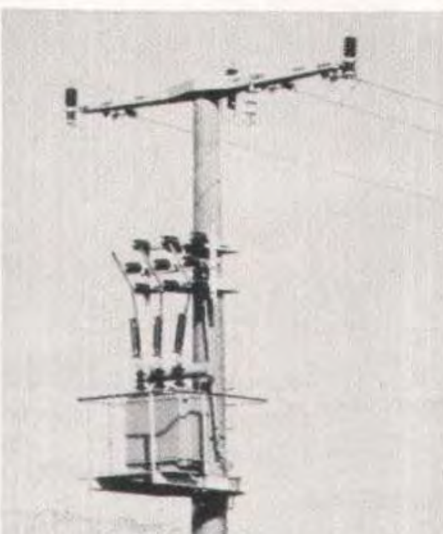


Fig. 4. Electric power distribution structure (Western Europe).

Fig. 5. Electric power distribution structure (Western Europe).

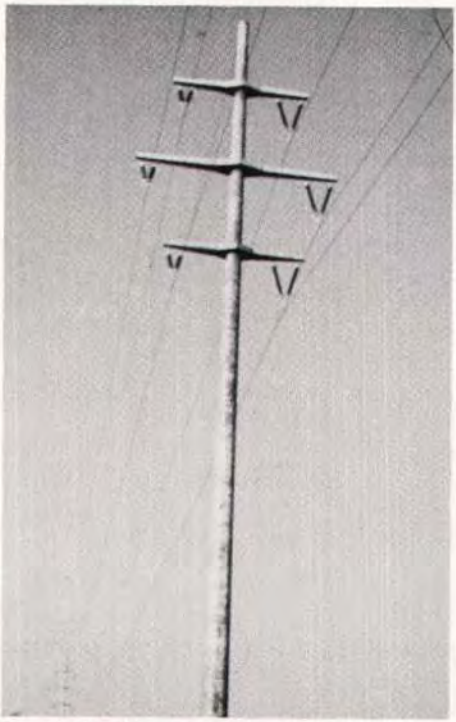


Fig. 6. Electric power transmission structure (Western Europe).

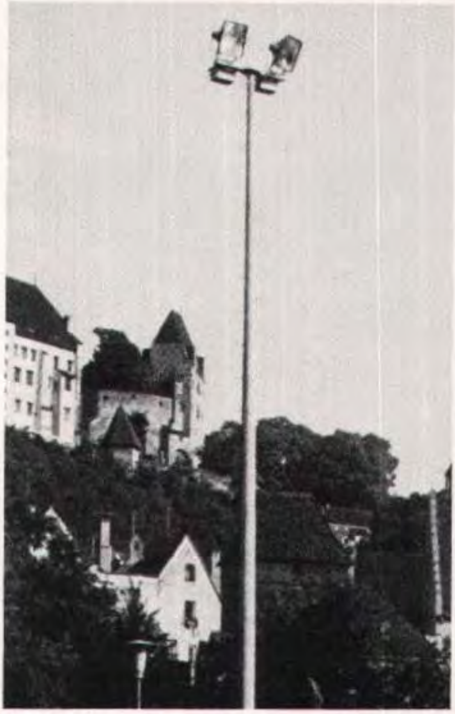


Fig. 7. High rise lighting structure (Western Europe).

Fig. 8. Street lighting structure (Western Europe).

Fig. 9. Street lighting structure (Western Europe).

106,000 units a year in 1959, including some 39,000 prestressed units. Since then, in order to use less steel but still increase the quality of the poles, the production of prestressed poles has gone up and by 1964 they displaced ordinary reinforced concrete poles completely. The annual production of poles for automatic signaling, telephone systems, and overhead electric power transmission for railroads is over 100,000 units.

In Japan there is a nationwide use of prestressed concrete poles for electric power transmission, distribution and substations, overhead power transmission for railroads, telephone and communication systems, lighting standards, flood-lighting, and wire netting supports. In recent years Japan has become an exporter of large quantities of poles to all of southeast Asia, in addition to the west coast of North America (Figs. 11-14).

Prestressed concrete poles have also been made and used for power distribution in New Zealand, Australia, India, and South Africa since the mid 1950s.

In the United States reinforced concrete poles were used by some electric utilities in the 1930s, but the first prestressed concrete poles were used by Florida Power Corporation in 1954 when they designed a 66 kV pole and an H-section to use in a 110 kV H-frame structure. By the early sixties, Florida Power Corporation and Florida Power and Light Company were using prestressed poles for lighting and power distribution. Today, the Florida utilities are still using statically cast square prestressed poles for lighting, distribution, and for some transmission up to 230 kV (Fig. 15).

In 1962, the Eugene Water and Electric Board of Eugene, Oregon, constructed 1 mile (1.6 km) of double-circuit H-frame 115 kV transmission line along the McKenzie River, using a tapered I-shaped prestressed concrete pole as an aesthetic challenge. They have continued to use the I-shaped prestressed pole as a single circuit pole, and as a single and double circuit H-frame transmission structure (Fig. 16).

Fig. 10. Communication structure (Western Europe).

In 1964, the Virginia Electric and Power Company (Vepco) built its first prestressed concrete pole structures, a tapered I-shaped pole (Fig. 17). These poles were used to rebuild and upgrade, from 34.5 to 115 kV, a 3-mile (4.8 km) water crossing in the coastal area of North Carolina. Each pole was set in a post-tensioned, centrifugally-spun concrete cylinder pile of predetermined

Fig. 11. Railroad electrification structures (Japan).

Fig. 12. Electric power distribution structures (Japan).

length. The piles were jetted and driven into the bottom of the Currituck Sound. Vepco in 1966, used a static cast, tapered square pole for lighting distribution and some 115 kV transmission.

In 1968 Vepco changed from statically cast poles to centrifugally-spun prestressed poles having tapered, hollow, circular cross sections. This type of pole is still being used for lighting, distribution, transmission, and substation structures.

Currently, there are several utilities in the United States and Canada using a number of different types and shapes of prestressed concrete poles for lighting and for power transmission and distribution.

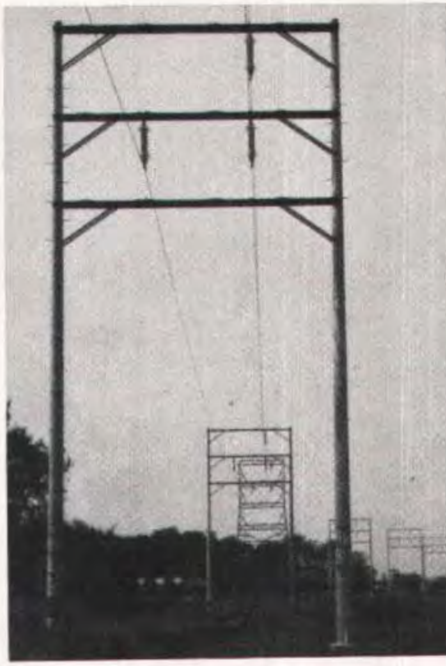


Fig. 13. Power transmission structures (built in Japan).

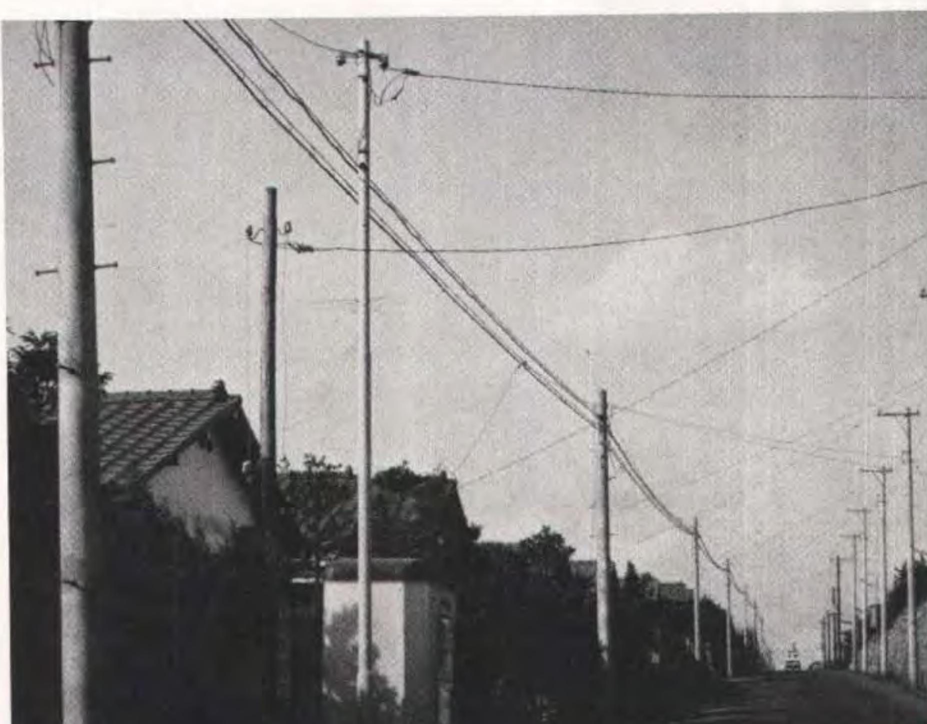


Fig. 14. Telephone line structures (Japan).

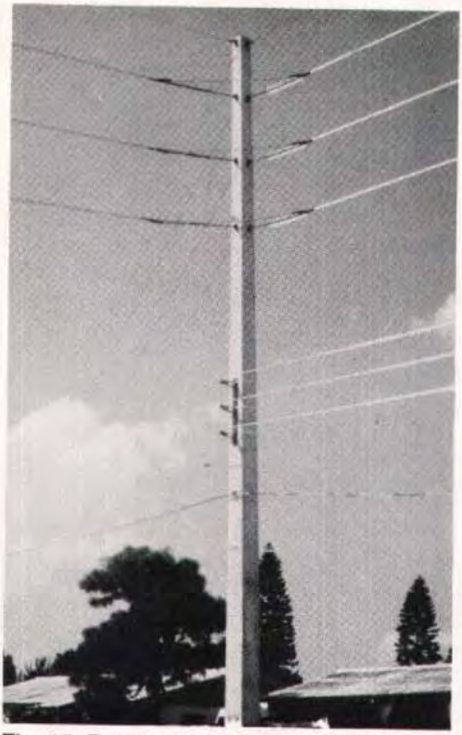


Fig. 15. Power transmission structure (Florida Utility Company).

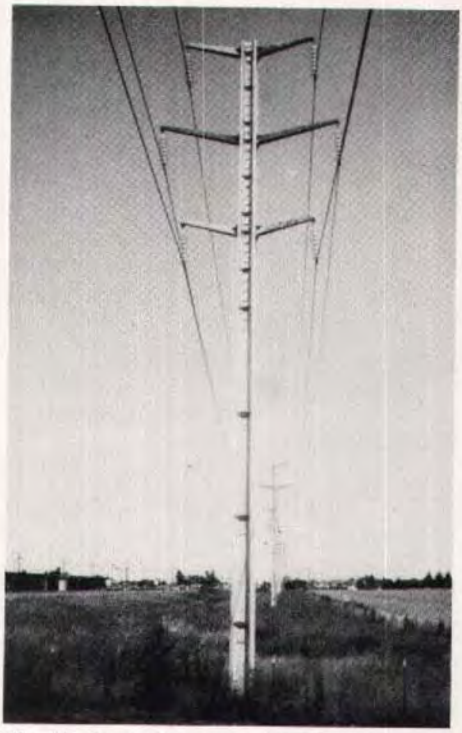


Fig. 16. Power transmission structure (Eugene Water and Electric Board).

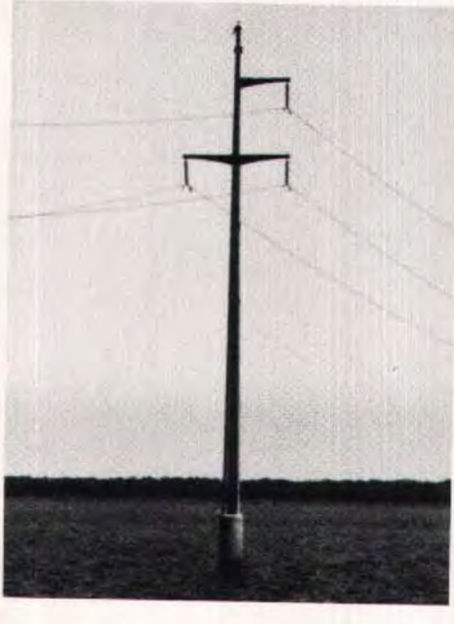


Fig. 17. Power transmission structure (Virginia Electric and Power Company Currituck Sound Crossing). Built in 1964, these tapered I-shaped prestressed concrete poles were used to upgrade a 34.5 to 11.5 kV, 3 mile (4.8 km) water crossing in North Carolina. The poles were manufactured using the centrifugally-spun cylinder pile method.

DESIGN LOADINGS

Pole structures have to be designed to be reliable, serviceable, and to resist (without permanent distortion) all anticipated maximum service loads. In the United States, the loading conditions for electric power structures have to meet or exceed those given in the National Electrical Safety Code (NESC). For lighting structures, the loading conditions have to meet or exceed the NESC and/or AASHTO Standard Specifications for Structural Support for Highway Signs, Luminaires, and Traffic Signals. For microwave or antenna towers, the loading conditions have to meet or exceed the Electronic Industries Association Standard RS-222-C.

These loadings are minimum requirements and the designer's study of the structure location will determine whether these minimum requirements should be increased. Loads other than those required by the codes or standards that affect the design of these structures are those due to extreme weather, or to manufacturing, handling, transportation, and erection.

Electric Power Transmission Structures

In these types of structures the design loadings are due to (1) vertical loads, (2) transverse loads, and (3) longitudinal loads.

In addition to their own weight, structures are subjected to vertical loads due to dead loads of wire and attachments, and ice. The only variable here is the ice. A structure should be designed for vertical loads due to maximum vertical span, ice, construction, and maintenance loads.

The maximum vertical span depends on the terrain. Structures located on hills must support more wire load, low point to low point of wire sag, than the adjacent structures which are located on level ground (Fig. 18). The horizontal wind span is unaffected by terrain and will always be half the true horizontal distances between suspension structures.

Selection of the ice loads should be based on experience records for each service area; these records can be obtained from the U.S. Weather Bureau. Ice build-up may not be the same on all spans, and the engineer should recognize this effect when designing for unbalanced longitudinal loads.

During the wire stringing operation, the structure may receive vertical loads more severe than those in normal design loadings.

For maintenance loads, the structure should be designed to provide adequate support for conductors that are being lowered during a repair operation. This is often overlooked and could be a critical factor for a light structure.

Transverse loads are caused by (1) wind pressure on the structure and wires and (2) the transverse component of line tension at angles. In combining these loads to give a total transverse design loading, it is important to ensure that the appropriate conductor tension be used with the corresponding wind load.

Extreme wind forces on transmission lines may be caused by three fundamentally different types of meteorological systems: (1) tornadoes, (2) hurricanes, and (3) local thunderstorms.

A tornadic wind is a type of a whirlwind with rotating wind velocities estimated at well over 200 mph (322 km/h). Tornadoes have velocities and accompanying pressures so great that they destroy everything before them, but fortunately they take narrow paths and, as a rule, do not last long. Reported losses of transmission structures in a tornado are usually small, typically only one to six structures for each strike. Engineers generally agree that it is uneconomical to design structures to resist tornadoes.

A hurricane is a very violent

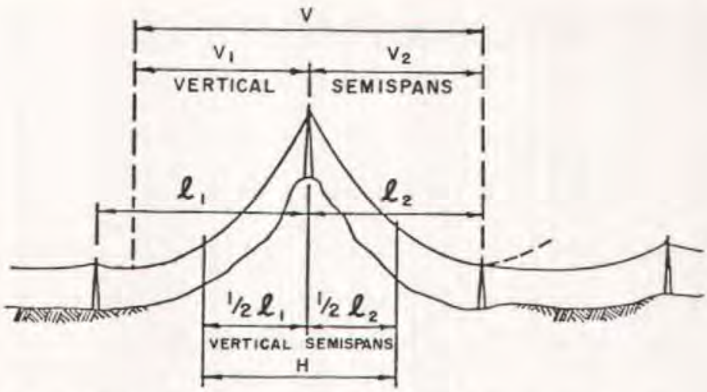


Fig. 18. Vertical and horizontal design spans for pole structure.

windstorm out of the West Indies; these windstorms usually blow up in summer and fall of the year. On the Beaufort scale, winds of over 75 mph (121 km/h) are classed as hurricane winds. The storm usually starts as a tropical depression in the Atlantic Ocean. As it moves along its path, the counter-clockwise winds around the center grow in intensity in the area covered. Winds of 100 to 120 mph (161 to 193 km/h) are common. Hurricanes cause great destruction each year to parts of the Caribbean Islands, Gulf, and East Coast States. In the Pacific Ocean, these storms are called typhoons. Engineers in the areas affected by hurricanes have to pick a design for extreme wind speed that is a balance between risk of failure and structure cost.

Thunderstorms and squall lines are more localized and random in their impact and are the main concern of engineers in most parts of the country.

To determine the design wind load, the engineer needs to know a design wind velocity and a suitable equation by which it can be converted to pressure on the transmission line. The design wind loads may be determined for a geographical area by using the annual extreme fastest-mile wind charts developed by the U.S. Department of Commerce—Environmental Science Service Administration using statistical probability.

Charts are available for elevations of 30, 60, and 90 ft (9, 18, and 27 m) above ground, in varying mean-recurrence intervals. To realize what these charts are, consider the wind velocity record of Fig. 19. It represents a 5 minute wind variation at the location of a certain structure during the most severe storm of a given year.

The 5 minute average and the peak 2-second gust are self-explanatory. The fastest mile is defined as the average velocity at which 1 mile (1.6 km) of air passes the anemometer.

The operational life of the transmission line should determine the mean recurrence interval chart that is used. Utilities normally consider the life of a wood pole line at about 25 to 30 years, steel and prestressed concrete structural lines at about 60 years. It is recommended that a 50-year mean recurrence interval be used for prestressed concrete structure lines.

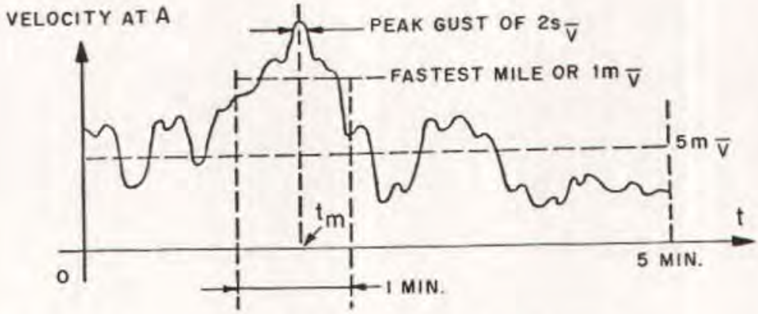


Fig. 19. Wind velocity strip graph for designing pole structure.

A well-known formula for increasing wind speed according to height is:

$$V_h = V_o \left(\frac{h}{h_o} \right)^{1/n} \quad (1)$$

where

V_h = adjusted velocity, mph
V_o = annual extreme fastest mile wind chart velocity, mph
h = adjusted elevation, ft
h_o = elevation of V_o, ft
n = constant dependent on surface roughness (varies from 2 to 7)

A value of 7 for n is normally used for level or slightly rolling land with scattered trees or buildings. The formula is not reliable for use in decreasing wind speeds below 30 ft (9 m). Wind velocity, in mph, may be converted into wind pressure by using the following formula:

For wind pressures on cylindrical surfaces:

$$P = 0.0025\, V^2 \quad (2)$$

For wind presures on flat surfaces:

$$P = 0.0042\, V^2 \quad (3)$$

where

P = wind pressure, psf, on projected area of surface
V = design wind velocity, mph

Wind Pressure on Structures

There appears to be general agreement that short wind gusts are large enough to envelop a transmission structure. For this reason, the design wind pressure on a transmission structure should be determined from a gust velocity. Relationships between peak gust and fastest mile have been developed; one such expression used by the industry is:

$$\text{Peak gust} = 1.3 \times \text{Fastest mile} \quad (4)$$

To find the design wind pressure on a transmission structure, the wind pressure formula should be adjusted to:

$$P = k\,(1.3V)^2 \quad (5)$$

where k is the surface shape coefficient.

The same problems of selecting wind velocity and suitable equations also arise with regard to the transmission line. It has been mentioned that a short gust of wind could envelop a structure, but it cannot envelop a span of transmission line approximately 1000 ft (305 m) in length. Field tests have shown that a reduction factor can be applied to modify the conventional drag coefficients in picking a wind pressure on a conductor. After the conductor design pressure has been determined, an effective wind span factor (f_{ews}) of 0.6 should be applied to all spans in excess of 1000 ft. For span lengths between 300 and 1000 ft (91 and 305 m), this factor will vary and should be determined as follows:

$$f_{ews} = 1.0 - 0.4\,(S - 300)/700 \qquad (6)$$

where S is the span length, ft.

For spans of 300 ft (91 m) or less, the effective wind span factor should be 1.0.

Extreme wind with heavy ice is another condition the engineer has to consider. Maximum winds that occur during sleet storms are usually lower in intensity than winds at other times in the same area. The engineer must review local weather records to determine the range of wind velocities that occur during freezing precipitation. Then, using the appropriate ice loading, he must calculate whether the wind-with-ice condition is more critical as a transverse load than the higher wind on bare conductors.

A study made in Canada of maximum wind recorded during freezing precipitation and for one day after, tabulated local variation. This technique is valuable for the engineer because data can be taken directly from weather records, thus showing winds which are likely to occur in a given area when there is ice on the conductor.

Transverse Loads and Line Angles

The transverse loading upon the structure must be taken as the resultant load equal to the vector sum of the transverse wind load and the resultant load imposed by the wires due to their change in direction. In deriving these loadings, a wind direction must be taken which will give the maximum resultant load. Proper reduction is made in loading to account for the reduced wind pressure on the wires resulting from the angularity of the application of the wind to the wires.

In areas of moderate to heavy ice loadings, this transverse component will tend to control the design of angle and dead end structures. In areas of no ice, or where only minor icing occurs, care should be taken in determining the effect of wind on line tension, being sure to use the same design wind velocity to calculate line tension as is used to calculate the strength of the structure. Care is necessary also in calculating which of these two "maximum" conditions is critical:

1. Maximum wind velocity with corresponding line tension
2. Maximum line tension with corresponding wind velocity

Longitudinal Loading

For years, transmission structures have been designed using a longitudinal loading condition of a broken conductor and/or overhead shield wire dead load. In recent years, the trend has been to use larger conductors, and many utilities have regarded the broken wire condition as being too conservative.

Longitudinal loads can be imposed on structures by many conditions other than broken wires. The following are several other conditions under which a structure will be subjected to longitudinal loadings:

1. Conductor or overhead shield wire stringing loads
2. Unequal spans
3. Wind parallel to the line
4. Wind at 45 deg to the line
5. Ice-unloading unbalance
6. Loss of an adjacent structure

During stringing operations there are instances where the stringing block may jam momentarily, or a sleeve passing through the block may "hang up," causing some longitudinal load to be applied to the structure. Stringing equipment located too close to a structure may impose detrimental vertical and longitudinal loads to the structure during wire stringing operations.

Table 1. Overload capacity factors for metal and prestressed concrete structures (NESC Table 261-2).

Strength	Overload capacity factors	
	Grade B	Grade C
Vertical strength	1.50	1.10
Transverse strength	2.50	2.20
Longitudinal strength		
At crossings		
In general	1.10	No requirement
At dead ends	1.65	1.10
Elsewhere		
In general	1.10	No requirement
At dead ends	1.65	1.10

Note: The factors in this table apply for the loading conditions of NESC Rule 250B. For extreme wind loading conditions, see NESC Rule 260C.

In mountains or very hilly areas, it is possible to have longitudinal loads caused by unequal spans. This is due to the difference in tension in the wires in adjacent spans resulting from unequal vertical loading and/or unequal span length.

Winds not only blow on the structures in the transverse direction, but in the longitudinal direction and at all angles in between. The structures should at a minimum be designed to withstand the wind plus gust which may envelop it in the longitudinal direction. Winds blowing at 45 deg to the line can exert longitudinal forces on the structure due to wind on the wires and wind on the structure itself. These longitudinal forces should be investigated.

Ice-unloading unbalance is now the most commonly used longitudinal loading. The dropping of ice from one or more wires in different combinations is used by a designer to provide torsional strength in addition to longitudinal strength in a structure.

Ice may build up on wires in only a few spans causing longitudinal loads. In most cases, ice builds fairly uniformly on all spans, and usually drops from one span before another. Ice frequently falls in large chunks when it starts to melt. As these chunks fall from one span, the swing of the insulator strings in a longitudinal direction is noted. This has led designers to use only a percentage of the ice-no-ice tension differential as the longitudinal loading. The longer the insulator string, the more the tension differential is reduced.

Under Section 25, the NESC sets the minimum weather loading condition for which a transmission line is to be designed. These minimum weather loadings must be values of loading resulting from the application of Rule 250-B — Combined Ice and Wind Loading or Rule 250-C — Extreme Wind Loading, whichever is the greater.

In the design of pole structures, the term "overload capacity factors" is interpreted to mean that the structure should support, without permanent set, the maximum loadings to which it will be subjected multiplied by the appropriate overload factors.

The general loading requirements set forth in Section 25 of the NESC are to be multiplied by overload factors set forth in Section 26, "Strength Requirements" depending on type of structure, to establish the design loads (see Table 1).

The minimum strength of any pole structure must be sufficient to with-

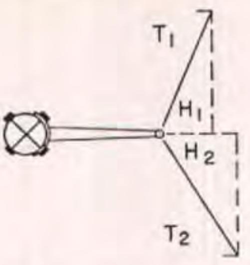


Fig. 20. Line angle loads acting on pole structure. Note: Line angle load = $H_1 + H_2$.

stand, without conductors, the extreme wind pressure in Rule 252 applied in any direction of the structure, multiplied by an overload capacity factor of 1.0. A gust factor appropriate for the wind pressure and structure height should be considered.

The overload factors for prestressed concrete pole structures as set forth in Section 26 of the NESC are shown in Table 1.

The transmission line designer, as can be seen, starts with the NESC recommended load schedule for geographic area in which the line will be built. Following this procedure, combinations of wind, ice and dead loads are added based on local weather conditions, past experience, and historical precedence. To each of the contingencies, overload factors are applied to weigh the importance of that particular contingency under the site's unique conditions.

The designer of the structure, must examine all these loading conditions, since different loads will control individual members or sections of the finished structure design.

For design purposes, all wire loads on a transmission structure are considered to be concentrated loads with a transverse, longitudinal and vertical component.

The following loads are typical of those usually considered:

1. Vertical Loads

A. Weights of bare wires plus insulators at attachment points
B. Weights of wires coated with ½ radial in. (13 mm) ice plus insulators at attachment points
C. Weights of wires coated with 1 in. (25 mm) radial ice plus insulators at attachment points
D. One man plus equipment at critical points
E. Vertical components of stringing loads

Loads A, B, and C are calculated as the product of the weight of wires (coated or not) per unit length by the vertical span V.

2. Transverse Loads (Fig. 20)

F. Line angle loads from bare wires subjected to extreme wind, fastest mile at 60 F (16 C).
G. Line angle loads from ½ in. (13 mm) ice coated wires subjected to 40 mph (64 km/h) wind at 0 F (−18 C) [4 psf (0.19 kPa) wind prssure]
H. Line angle loads from wires coated with 1 in. (25 mm) ice at 0 F (−18 C)
I. Transverse force on structure due to 40 mph (60 km/h) wind [6.4 psf (0.3 kPa) wind pressure]
J. Transverse force on structure due to extreme wind (say 1.3 × fastest mile)
K. Transverse forces at attachment points due to 40 mph (64 km/h) wind on wires coated with 1 in. (25 mm) ice
L. Transverse forces at attachment points due to extreme wind (say 1.3 × fastest mile) on wires
M. Transverse component of force on structure due to diagonal extreme wind

Loads F, G, H, and I can be determined as algebraic sums of the transverse components of tensions of attached wires.

Note that transverse forces K and L are the products of wind forces per unit length of wire by appropriate wind spans.

3. Longitudinal Loads

N. Unbalanced wire tensions due to unequal vertical loads, different span configurations, etc.

O. Longitudinal force on structure from extreme wind (say 1.3 × fastest mile)

P. Longitudinal component of force on towers from diagonal extreme wind

Q. Longitudinal component of stringing and construction loads

R. Broken wires — Loads should reflect experience under actual conditions

S. Ice-unloading unbalance — Ice-no-ice tension differential × percent reduction due to insulator swing

T. Dead-end tensions from bare cables subjected to extreme wind at 60 F (16 C)

U. Dead-end tensions from ½ in. (13 mm) ice coated wires subjected to 40 mph (64 km/h) wind at 0 F (−18 C)

V. Dead-end tensions from 1 in. (25 mm) ice coated wires at 0 F (−18 C)

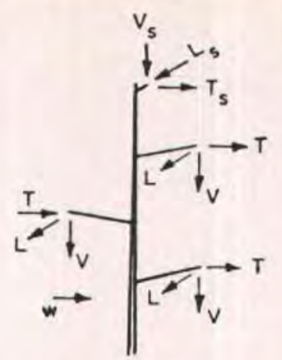
Design loads for prestressed concrete structures are ultimate loads. The following design loading conditions, load multiplied by overhead capacity factor, can be used at overhead shield wire and conductor points:

NESC — Heavy Loading

Case I 1.50 (B) (Vertical load)	2.50 (K) + 1.65 (G) (Transverse load)

Extreme wind

Case II 1.50 (A) (Vertical load)	1.1 (L) + 1.1 (F) (Transverse load)



T = transverse load
L = longitudinal load
V = vertical load
w = wind on structure
V_s, L_s, T_s = shield wire loads

Fig. 21. Design loads ("load tree") acting on pole structure.

Heavy Ice

Case III 1.50 (C) (Vertical load)	1.1 (H) (Transverse load)	1.1 (S) (Longitudinal load)

The above are the basic loading cases. This is the procedure the utility engineer goes through in supplying the designer with the "load trees" (Fig. 21) required to design prestressed concrete pole structures.

Lighting Structures

Lighting pole structures must meet or exceed the design loadings of the NESC and/or AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals, and include (1) dead load, (2) live load, (3) ice load, and (4) wind load.

In addition to its own weight, the dead load includes all permanently attached fixtures, including hoisting devices and walkways provided for servicing of luminaires, if required.

Live load need not be applied to structural supporting members. The only member requiring a live load design would be the walkway and ladders.

The ice load of 3 psf (0.14 kPa) is applied only to the attached fixtures, ladders, walkways, luminaires or signs, and is based on 0.50 in. (13 mm) of ice at 60 pcf (961 kg/m³).

Wind speeds are based on the 25-year mean recurrence interval wind speed maps developed by the U.S. Department of Commerce — Environmental Science Service Administration and adjusted for height. Wind pressure on structural supports is computed from:

$$P = 0.00256\,(1.3V)^2\,C_d C_h \qquad (7)$$

where

C_d = a dimensionless drag coefficient that varies with the shape of the support structure receiving the wind loading

C_h = coefficient of height, derived from $(H/30)^{1.7}$

H = height at which pressure is determined

V = wind speed at 30 ft (9 m) height

1.3 = gust factor

The wind load on the structure must be determined by the areas of horizontal and vertical supports, luminaires and signals and must be applied to the surface area seen in normal elevation.

Antenna Support Structures

Antenna support structures must meet or exceed the design loads of the Electronic Industries Association, EIA Standard RS-222 latest revision, and include (1) dead load, (2) ice load, (3) wind load, and (4) limit values of twist, sway, and displacement.

In addition to its own weight, the dead load includes all permanently attached fixtures, antenna assemblies, transmission lines, reflectors, conduits, lighting, climbing facilities, platforms and signs.

The ice load is based on solid ice [density of 56 pcf (897 kg/m³)] applied to the attached fixtures. This standard does not specifically state an ice thickness required. Consideration to its location and the climatic conditions should be given to an ice thickness requirement in preparing the specifications.

Wind loads are defined as the force and torques produced by a specified unit horizontal wind pressure acting on the tower, antenna assemblies, reflectors, and other fixtures attached. In all cases, the specified ice coating should be included as part of the projected area. Minimum horizontal wind pressure in pounds per square foot (psf) are referred to on a map chart called "Wind Loading Zones" which is part of the standard. For towers under 300 ft (92 m), Zone A — 30 psf (1.4 kPa); Zone B — 40 psf (1.9 kPa); Zone C — 50 psf (2.4 kPa); the pressure is assumed to be applied uniformly over the entire height of the structure.

Tower twist is defined as the horizontal angular displacement of the tower from its no wind load position at that elevation. Tower sway is defined as the angular displacement of a tangent to the tower axis at that elevation from its no wind load position at that elevation. Tower displacement at any elevation is defined as the horizontal displacement of a point on the tower axis from its no wind load position at that elevation. A table of allowable twist and sway values for microwave tower-antenna-reflector systems is attached to the standard.

Shape

The shape of poles has a bearing on the design and manufacturing technique. The normal sections adopted in various countries are shown in Fig. 22. The simplicity of the square and rectangular solid section poles of small

Fig. 22. Typical cross sections of prestressed concrete poles.

length, up to 40 ft (12 m), make them easy to manufacture, facilitate positioning of steel at corners giving maximum resisting moment for a given depth, and occupy less space in transportation. Circular hollow sections seem to be ideal for the longer poles.

Cylindrical hollow poles have these advantages: less weight; equal strength in all directions eliminating any special care during handling, transportation and erection; a denser and higher quality of concrete from the spinning process. The absence of corners and the smooth, dense minimum surface area give greater protection against corrosion failure. However, there is a considerable divergence of opinion concerning the best shape of a pole.

Vierendeel poles are also widely used. Their use has been in structures of 33 ft (10 m) in length and above and where higher transverse loads are involved. They have the disadvantages of having larger exposed areas and thin elements, leading to possible concrete cracking and corrosion. It should be noted that if the horizontals are not spaced further apart, they will be misused for climbing. East European countries have used the Vierendeel pole in a "Pi" structure or H-frame structure for carrying transmission lines.

DESIGN CRITERIA

Design criteria for poles vary significantly from country to country. However, poles generally act as a cantilevered structure, and should be designed and/or analyzed as a tapered member with combined axial and bending loads. The shear forces are very small when compared to the bending moments; the deflections are relatively large, and this helps to provide the resilience. The axial loads, being small, are generally ignored except when the structure is guyed.

It is essential in the design to examine the stresses induced by handling, transportation and erection. Under severe conditions of handling, these may exceed in-service stresses. The stresses imposed during transportation will depend on the method of transportation available. The common lifting points used are the third points or, in some cases, the center of gravity. Erection stresses will likewise depend on the point at which the pole is lifted. In considering the latter, the weight of the cross-arm and/or other attachments should not be overlooked in calculating the position of the center of gravity.

The concrete structure should be proportioned so that the deflection due to the service loads will not be detrimental to the strength, serviceability requirements, and the aesthetic qualities of the structure.

The pole has to withstand equal bending moments in opposite directions and therefore concentric prestressing is provided. Hence, the magnitude of the prestress can only be about one-half of the value that can be used for bending in one direction. This is an important difference in the design of prestressed concrete poles compared to other types of prestressed structures.

The ultimate moment capacity of the pole at various sections is a function of the strains of prestressing steel and concrete and the effective stresses in the prestressing steel. Loaded to failure, the pole will fail in one of the following modes at the section which undergoes a higher ratio of bending design moment to ultimate moment capacity.

1. **Rupture of steel** — The pole might have one or more sections having a low percentage of steel, i.e., under-reinforced sections. The ultimate strength of the steel is attained before the concrete has reached a highly plastic state.
2. **Crushing of concrete** — The pole might have one or more sections having a high percentage of steel, i.e., over-reinforced sections. The steel stresses do not exceed the yield point, and failure results in the crushing of concrete.
3. **Failure of both steel and concrete** — Sections of the pole may have a balanced behavior of the two materials. The steel would be stressed into the plastic range and the concrete would attain the maximum strain defined by its capability.

To achieve the balanced behavior in the third failure mode in a tapered pole presents a practical problem regarding the application of prestress. However, systems have been developed to reduce the effective prestress in the upper portions of the tapered pole by preventing bond or by dead-ending some of the tendons along the length of the pole.

Another design factor is the flexural moment that causes initial cracking on the tension face of the pole. This is the point at which all the prestress and the tension capacity of the concrete has been used. This has been found to be about one-half of the ultimate moment of the fully prestressed sections. The design permits the pole to exceed the cracking moment, but not to fail under ultimate design conditions.

So far the paper has concentrated upon fully prestressed concrete poles, but some of the larger poles used for transmission line structures in Europe and India are partially prestressed. The definition of each follows:

Full Prestressing — A concrete structure is fully prestressed if the stresses due to bending perpendicular to the direction of prestressing calculated for the full design service load are compressive. (It should be noted that full prestressing does not provide absolute safety against tensile stresses or cracking due to shear, torsion, temperature effects or imposed deformations.)

Partial Prestressing — A concrete structure is partially prestressed if substantial tensile stresses or cracks perpendicular to the direction of prestressing can occur in the concrete under the full design service load. Such cracks may be of limited width to satisfy durability or appearance purposes. (In these cases, additional mild steel reinforcement is added in the direction of the tendons to meet ultimate strength requirements.)

Reinforced Concrete — A reinforced concrete structure is one in which none of the steel reinforcement is prestressed during construction.

The full design service load is the equivalent of the design loading before the applicable code overload capacity factors have been applied.

Partially prestressed poles will contain both prestressing tendons and mild steel reinforcement. The reinforcement should have high bond characteristics. The degree of prestressing will influence the behavior of the structure under service loading with regard to deflection, tensile stresses and cracking, and all of the steel will operate to provide an adequate factor of resistance at the ultimate stage.

The service load should be considered in two parts:

(a) That portion of the load which is permanently or frequently occurring.

(b) The maximum possible service load which may be applied.

In Condition (a) the prestress should be such that control is exercised in one or more of the following ways:

1. Control of cracking, either no cracks or crack width limited to a defined amount.
2. Control of concrete bending tensile stress in the section under load, either no tension or a defined maximum tensile stress transverse to the direction of prestressing. Stress created by secondary moments may also need to be considered.
3. Control of deflection, zero deflection or a defined maximum value, positive or negative, in relation to the span.

For Condition (b) it should be accepted that cracks occur and the member deflects more than under Condition (a) loading, but these cracks will close and the deflection will reduce on removal of the load. It must be ensured that the structure will return to a condition complying with the requirements for Condition (a) when the infrequent load is removed. Investigations at Condition (b), therefore, need only be such as to ensure that the structure returns to these conditions and perhaps that no excessive deflection will occur under this maximum load.

The design basis for partially prestressed poles consists, therefore, essentially of three stages:

1. Assessment of prestressing force to satisfy serviceability requirements under permanent or frequently occurring load.
2. Assessment of ultimate strength of member (in bending, shear, etc.).
3. Checking cracking and deflection conditions under the maximum possible service load.

It is necessary to control the extent of cracking at serviceability conditions in order to ensure durability of the steel against corrosion and to ensure an acceptable surface appearance. The degree of cracking which can be allowed must depend on the aggressiveness of the environment to which the structure will be subjected, and the quality of the concrete being used.

In designing prestressed concrete poles by the codes of different countries, a certain cracking resistance under sustained loads is specified, though the loads and requirements about the admissibility of cracks differ. The methods of controlling strength, rigidity, and the prestress in the steel and the concrete differ as well. In view of this, it is difficult to compare the efficiency of poles used in different countries.

In East Germany, poles are designed by the ultimate load method for State 2 to provide a prestressing value compatible with the allowable crack width under the average and maximum loads. The design is based on the regulations in TCL 0-4227 and TCL 112-0491.

In West Germany, the loads on poles, their interaction under various working conditions and the design methods are based on a number of documents in force, including: DIN 4228-1964, DIN 48353, DIN 1055, VDE 0210/5.62, and DIN 4227. Checks on stresses are made under average and exceptional loading cases. The standard design restricts the crack width to less than 0.1 mm (0.004 in.) with a spacing of about 100 mm (4 in.).

In the Soviet Union, poles are de-

Table 2. Comparison of various provisions for different codes of practice.

Poles	Czechoslovakia	Germany DIN 4227/4228	India IS 1678	Japan	New Zealand	United Kingdom BS 607
Load factor against crack	1.0	1.5	1.2	1.0 Crack width under design load 0.25 mm	—	1.2
Load factor against failure	2.0	1.75	2.5	2.5	—	2.5
Equivalent load applied at	10 in. (250 mm)	—	24 in. (600 mm) from top	10 in. (250 mm)	12 in. (300 mm)	24 in. (600 mm)
Embedment length	0.2 of height	0.2 of height	4 to 8 ft (1.2 to 2.4 m)	6.5 ft (2.0 m)	7.22 ft (2.2 m)	4 to 8 ft (1.2 to 2.4 m)
Concrete strength	7100 psi (50 MPa)	10,000 psi (70 MPa)	Above 6000 psi (42 MPa)	Above 4500 psi (38 MPa)	Above 5700 psi (40 MPa)	Above 6000 psi (42 MPa)

signed in accordance with the relevant standards and codes using the limit state method (SNiP II-C.1-62, SNiP II-1.9-62 and others). Design by strength, deformation as well as crack width are covered by this method. The first limit state, ultimate strength, must always be checked. The second limit state needs only to check the behavior of terminal poles under emergencies. The third limit state requires the behavior of every concrete member to be checked for normal service loads.

In the United States at this time, it is recommended that prestressed concrete poles be analyzed for ultimate strength in accordance with the basic provisions given in the Building Code Requirements for Reinforced Concrete, ACI 318-83, modified to include the effects of prestressing. Pole structures should be designed to withstand the maximum of the forementioned loading conditions, including the overload factors, without exceeding the ultimate strength of the pole. Under normal working conditions, the design should not exceed the cracking moment.

In Canada under the proposed new CSA Standard A14-M, Concrete Poles, poles may be designed in accordance with CSA Standard A23-3, Design of Concrete Structures for Buildings, without recourse to classification testing, provided that substantive information is available for presentation to verify that the poles, as manufactured, are in accordance with the design assumptions. Poles may also be designed using empirical coefficients obtained from classification tests, conducted in accordance with Clause 7 — Classification Testing Procedures and within some stated limitations.

Table 2 compares the major provisions of the various codes of practice and lists some of the basic code requirements.

It would be helpful to have some international standardization of design methods for pole structures fulfilling the same functions and operating under substantially identical conditions.

MANUFACTURE

The successful manufacture of prestressed concrete poles depends on the local conditions and available equipment. It can be done in specialized factories using the sophisticated spinning method with steam and/or water tank curing, or as in India, in remote areas on site locations, using portable pretensioning beds, local untrained labor, air curing and specially designed bullock carts for transporting and erecting the poles. The following are the principal manufacturing methods.

1. Industrial manufacturing of complete poles at specialized factories:
   (a) By centrifugal casting method with demountable steel forms;
   (b) By vibration, compacting on a bed with the molds laid horizontally;
   (c) By compaction in a machine with either transverse or longitudinal vibration.
2. Manufacture may be on site with poles placed horizontally.
3. Poles may be made from precast sections which are assembled on site by post-tensioning or by splicing, or may be partly preassembled in special casting yards.

In most of Europe and Japan, concrete poles are economically mass produced by well equipped plants using the centrifugal casting method. The basic manufacturing equipment needed is the spinning machine and the steel forms. The spinning machines are of heavy-duty, roll-bench type and have sets of spinning wheel assemblies at 10 ft (3 m) intervals. The spinning wheel assemblies are hard face, long wearing wheels mounted on extra large anti-friction rollers and ball bearings in totally sealed pillow blocks. The machine is normally driven by a full length

Fig. 23. Open from concrete placement (with reinforcement in place) used in manufacture of prestressed poles.

drive-shaft with ball type universal joints and a single drive station located at the middle of the machine. Some of the more modern machines are equipped with automatic form loader/ unloader for maximum efficiency.

The demountable steel forms usually consist of two halves, but some plants use a single piece form. These forms are made in 10 ft (3 m) sections and are bolted end to end to the required pole length. The forms are precision designed and built for ruggedness; they are statically and dynamically balanced for smooth, vibration-free running. Forms are available in a wide range of sizes, lengths and shapes. The usual shape is round with a uniform taper from top to bottom. Other shapes used are hexagonal, octagonal, square with chamfered corners, triangular with chamfered edges, and cylindrical. They are normally tapered to any dimension from 0.15 to 0.180 in./ft (12.5 to 15 mm/m).

The process of making a spun concrete pole is to lay out the oiled lower

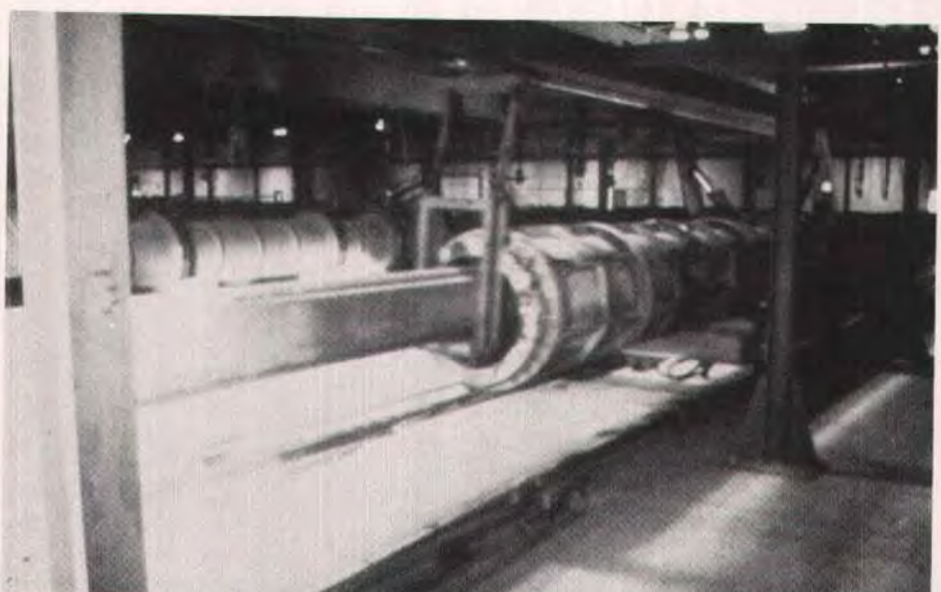


Fig. 24. Closed form concrete placement used in manufacture of poles.

Fig. 25. Closed form concrete placement used in manufacture of poles.

half of the form, to place full length spiral wire wrap, to pull reinforcing strand or wire through spiral wrap to anchor heads, then to chuck and apply a small amount of stress. In plants using "open form filling" (Fig. 23), the form is closed after it has been filled with a precalculated amount of concrete and final prestressing takes place after the form has been filled and fully assembled. In plants using "closed form filling" (Figs. 24 and 25), the form is closed immediately after placing the reinforcement, final prestressing takes place after

Fig. 26. Pole spinning operation.

Fig. 27. Pole manufacture in India using Hoyer's long line method.

the form has been fully assembled, but before a precalculated amount of concrete is pumped or dumped into it.

The form is then placed on the spinning machine where it is spun for several minutes. Two speeds are used in the process. At the lower speed, the mixture is divided uniformly along the form and the cylindrical cross section is formed. At the high speed, the tremendous centrifugal force created by spinning extracts excess water and consolidates the mix to an extremely dense, high strength concrete (Fig. 26).

After spinning, the form is taken to the steam curing area, where the pole is cured with low pressure steam for a period of time until the strength of the concrete in the pole has attained at least 3500 psi (24 MPa). The prestressed wire or strand is then released; the pole is dressed up and air cured for 28 days before shipping.

The Swiss BBRV prestressing system is also being applied to the manufacture of poles. In this system high tensile wire is anchored at the pole ends by means of button heads and special anchor washers. Using this system and some special auxiliary equipment, spun concrete poles are being made for electric power lines.

Vibration techniques are widely applied in the manufacture of concrete poles. In East Germany, a vibration method known as Mensel's method is used where lightweight horizontal molds are carried on mobile frames strong enough to take the tensile stresses of the prestressing steel. A production line system is used, and the molds and equipment circulate so that the workers do not have to move around. The production system embraces some of the processes used with other methods of production and includes one special feature. A formwork core occupies the space which will be the inner cavity of the pole, and this is rotated a little after the concrete has begun to harden. It is removed when the concrete has fully hardened. With this method of production, curing is done by a heat treatment cycle. The temperature of the poles is raised to 163 F (73 C), held and then cooled during a 24-hour period.

The most common casting method for solid sections (square, rectangular, channeled, I or Y shaped) is the long line method. In India it is called Hoyer's long line method. The forms are placed end to end along the length of the bed, 300 to 400 ft (90 to 120 m), with the narrow ends of the tapered poles facing each other and the wide ends next to each other (Fig. 27). The prestressing wire or strand is positioned by means of the holes in the bulkheads and is pretensioned against end abutments. The concrete is then fed into the forms and compacted with external vibrators operating at about 6000 cycles per minute.

In India steam curing is not generally adopted. The design of the concrete mix is such that the stress at transfer is obtained at the end of 3 days when wires or strands are cut and the mold released. In other countries the forms are covered and steam cured for about 24 hours, with a concrete release strength of 4000 psi (28 MPa). Poles made by the long line method can be made in any precast concrete yard, or on site.

In the United States today, the limited number of prestressed concrete poles being made for lighting and for power transmission and distribution are made by the long line method or the centrifugal casting method. Currently, in Canada all plants are making prestressed concrete poles by the centrifugal casting method.

MATERIALS

The strength of concrete and prestressing steel varies in different countries. Specified values for various countries are shown in Table 3.

In general, prestressed concrete poles are made from dense concrete with a 28-day strength of 3000 to 8500 psi (21 to 59 MPa). Some poles have been made with 10,000 psi (69 MPa), and in the United States one supplier is using 12,000 psi (83 MPa) concrete.

Tendons for prestressed concrete poles are usually one of the following: High strength, cold-drawn or heat-treated deformed wire with circular or oval sections; seven-wire strands of a smooth round wire; bundles of several similar wires; deformed bars made of a hot-rolled low-alloy or heat treated steel.

The mechanical properties, type and classes of steel vary from country to country and should be determined by appropriate regulations, standards and technical specifications (Table 3).

TESTING

Two types of testing are used to determine the flexural behavior and flexural capacity of poles under static loading conditions: pole testing and structure testing.

Pole testing is used to verify the design and quality of production of the poles. A test frame such as that shown in Figs. 28 and 31 is used. Here, the butt of the pole is fixed and the pole is pulled from a point below the tip, usually 2 ft (0.6 m), about a reaction point groundline distance from the butt. By adding the load in increments of the ultimate design, the cracking moment and physical behavior of the pole to destruction can be checked. It is good practice to include in the specifications some random pole testing to a percentage of the ultimate strength of the pole as a quality assurance check. A pole can be tested

Table 3. Materials used for manufacturing prestressed concrete poles.

	Concrete		Prestressing steel			Nonprestressed steel	
Country	Compressive strength, psi (MPa)	Type	Type	Ultimate strength, psi (MPa)	Section and shape	Type	Compressive strength, psi (MPa)
Czechoslovakia	7100 (50)	Normal concrete	ST 140/175	250,000 (1716)	$\frac{1}{4}$ in. (6 mm) Round		
France	5500-8500 (40-60)	Normal concrete	ST 140/160	230,000 (1569)	Plain round		
East Germany	6400-8500 (45-60)	Normal concrete	ST 140/175	230,000 (1569)	0.03-0.062 $in.^2$ (20-40 mm^2) Deformed oval	Deformed ST III	57000-71000 (392-490)
West Germany	4200-10000 (30-69)	Normal concrete	ST 140/175	230,000 (1569)	0.03-0.06 $in.^2$ (20-40 mm^2) Deformed oval	Deformed ST III	57000-71000 (392-490)
India	6000-7500 (42-52)	Normal concrete	Indented and plain round	230,000-255,000 (1569-1765)	$\frac{1}{8}$-$\frac{3}{16}$ in. (3.25-4.5 mm) Indented and plain round		
Japan	7100 (88)	Normal concrete	J15G3109	193,000 (1334)	$\frac{5}{16}$-$\frac{5}{8}$ in. (8-16 mm) Round deformed		

Table 3 (cont.). Materials used for manufacturing prestressed concrete poles.

	Concrete		Prestressing steel			Nonprestressed steel	
Country	Compressive strength, psi (MPa)	Type	Type	Ultimate strength, psi (MPa)	Section and shape	Type	Compressive strength, psi (MPa)
New Zealand	5500 (38)	Normal concrete	Hard drawn high tensile steel wire	192,000 (1373)	3/16-1/2 in. (5-12.7 mm) Round		
Poland/Romania	3000-8500 (20-60)	Normal concrete	ST 140/160	230,000 (1569)	Plain round		
United Kingdom	6000 (42)	Normal concrete	Plain and hard drawn steel wire	213,000 (1471)	1/8, 3/16, and 5/16 in. (3.25, 4.5, and 7 mm)		
Soviet Union	3000-9000 (20-60) 4260-5100 (30-40)	Normal concrete Lightweight concrete	Vr-II V-II	227,000-241,000 (1569-1667) 213,000-241,000 (1471-1667)	5/32, and 3/16 in. (4 and 5 mm) Round deformed 3/8 and 9/16 in. (9, 12, and 15 mm) 7-wire strand	Hot rolled deformed bars of low stressed steel are also used for prestressing. All longitudinal steel is tensioned	
United States	>5000 (35)	Normal concrete	ASTM A416	240,000 and 270,000 (1657 and 1863)	3/8, 1/2, and 7/16 in. (9.5, 12.7 and 11 mm) 7-wire strand		

Fig. 28. Horizontal testing of concrete pole for flexural capacity.

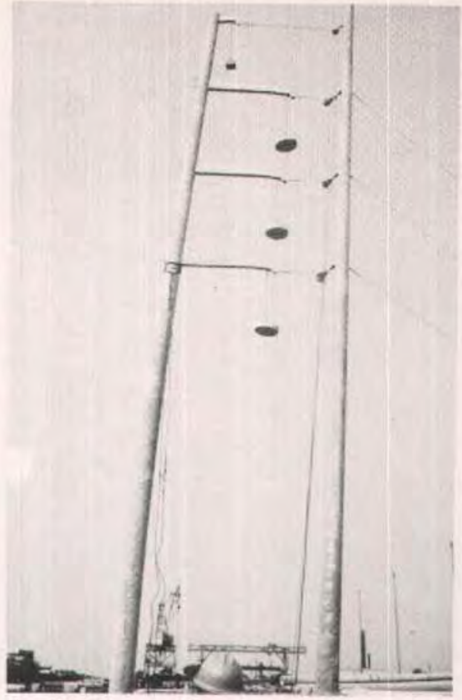


Fig. 29. Structural testing of prestressed concrete pole structure.

Fig. 30. Testing of pole structure.

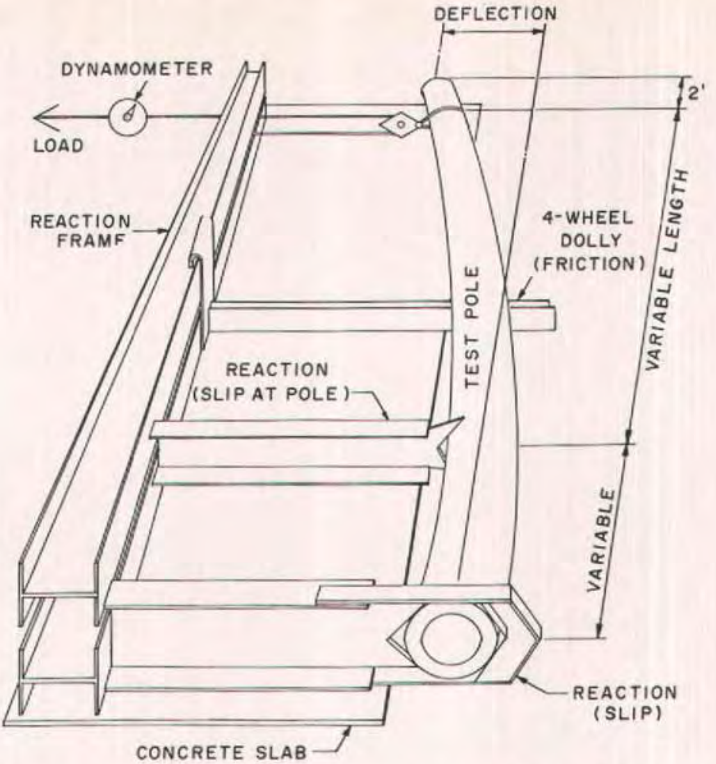


Fig. 31. Schematic of horizontal testing of concrete pole for flexural capacity.

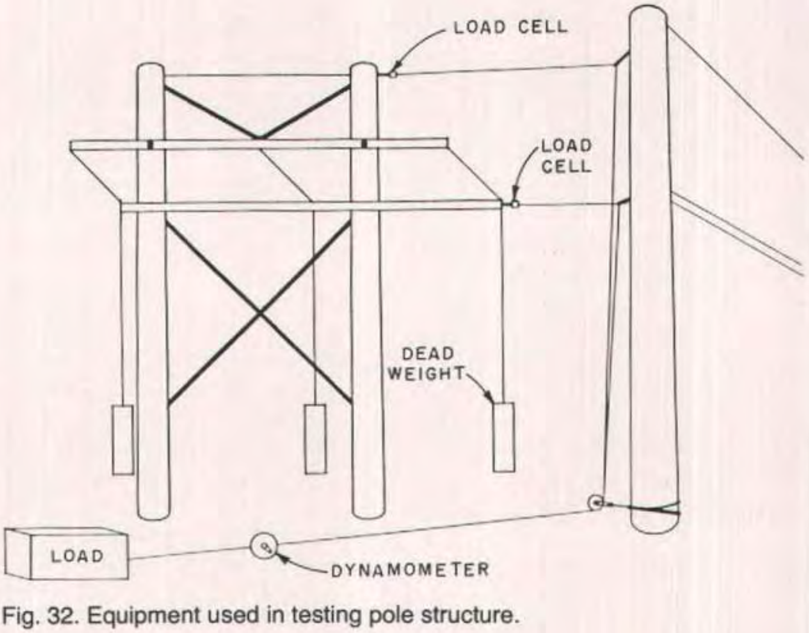


Fig. 32. Equipment used in testing pole structure.

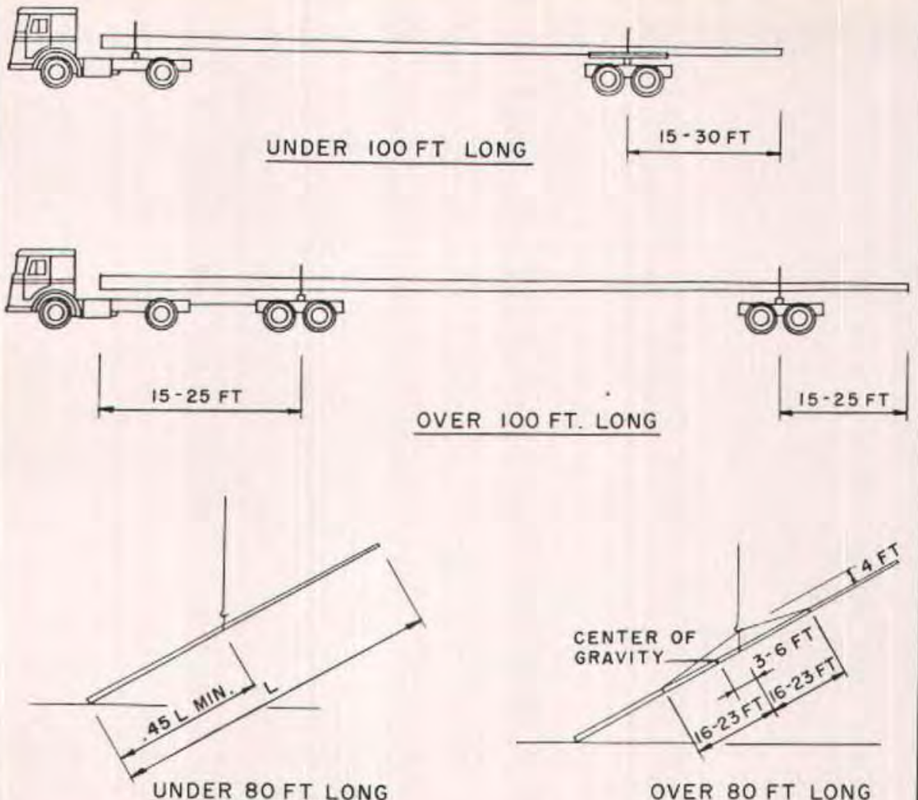


Fig. 33. Truck loading data and handling lifting points of pole.

up to 90 percent of ultimate, and when the load is released, show no effect of the loading.

Structure testing is the simulation of the structure as it is to be used, loaded in increments to its design loadings, in a static condition (Figs. 29, 30, 32), to check its structural strength and behavior.

HANDLING TRANSPORTATION AND ERECTION

Although prestressed concrete poles are resilient and resist cracking, they require special care in handling, transporting and erecting. Some guidelines for safe handling of prestressed concrete poles are:

1. Always handle prestressed concrete poles with the major axis in the horizontal direction, with a two-point balanced pickup.
2. While unloading from trucks, railroad cars or waterway barges, handle the poles gently. Under no circumstances should they be thrown onto a pile (Figs. 33 and 34).
3. Poles should be stacked level and supported so their own dead weight will not cause them to sag.
4. While transporting by pole trailer

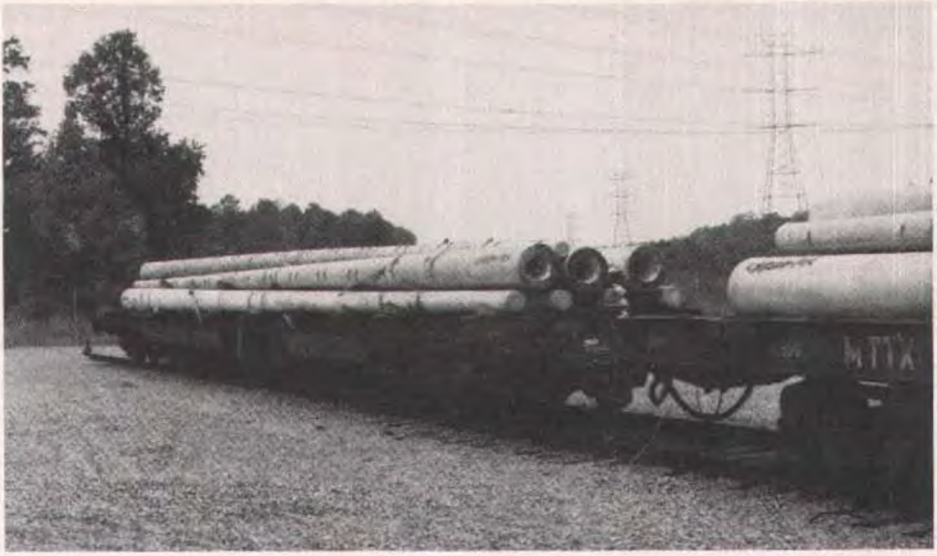


Fig. 34. Railroad transportation of poles.

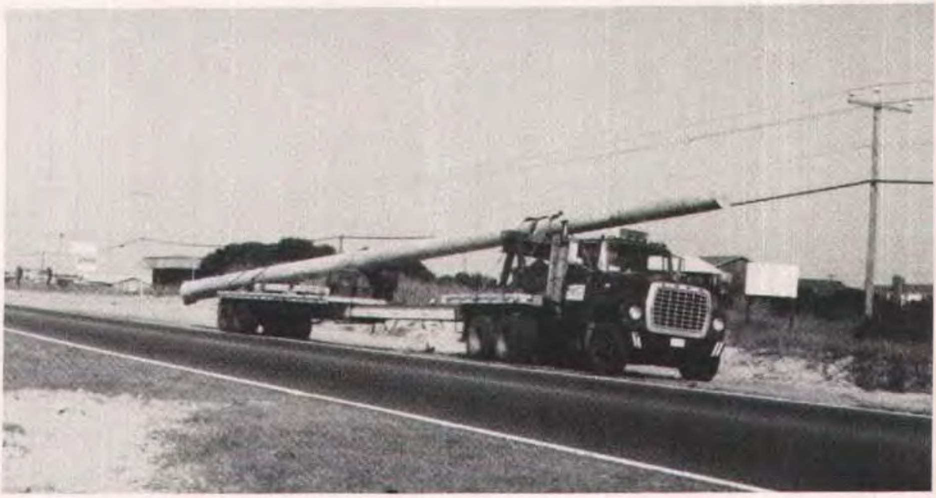


Fig. 35. Truck transportation of pole.

(Fig. 35), the poles should be held as rigid as possible to keep them from oscillating, which could cause them to crack. The use of a strongback is suggested, if necessary.

5. When erecting single poles, it is suggested that they be rigged as shown in Figs. 36 and 37.
6. H-frame structures should be assembled and lifted with the use of a spreader bar. With structures over 80 ft (24 m), a second pick point should be used to get the pole butts off the ground when lifting the structure to the vertical position (Fig. 38).

Fig. 36. Erection of a single pole structure.

Fig. 37. Erection of a German two section bolted spliced structure.

Fig. 38. Erection of an H-frame structure.

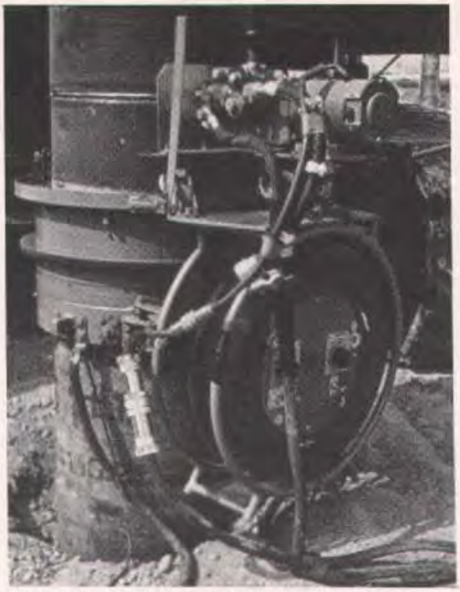


Fig. 39a. Full automatic welding apparatus (Japanese).

Foundations for prestressed concrete pole structures will vary because of groundline moment capacity. For most concrete pole applications, both self-supporting and guyed poles, direct embedment is all that is needed. When the range of 500,000 ft-lbs (68,000 N.m) groundline moment capacity is reached, the foundation requirements, depending on the reliability required of the structure, should be considered. Oversized holes may be excavated and controlled backfill of crushed stone, soil cement or concrete may be compacted around the embedded section of the pole.

When the concrete pole structure exceeds 1,000,000 ft-lbs (1,356,000 N.m) of groundline moment capacity, the foundation should be treated as a structural foundation and the capacity of the soil in designing the foundation becomes an

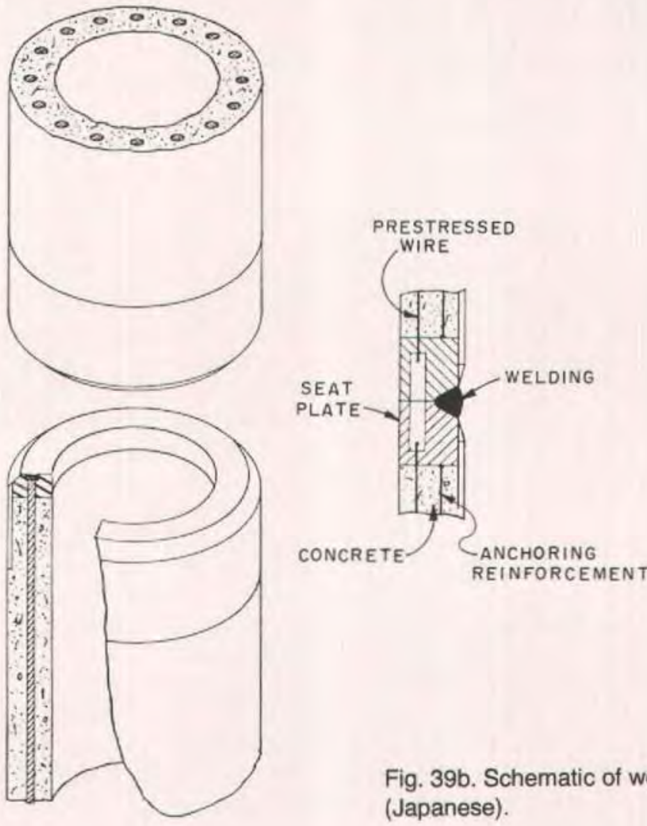


Fig. 39b. Schematic of welded splice (Japanese).

Fig. 40a. Bolted splice (German).

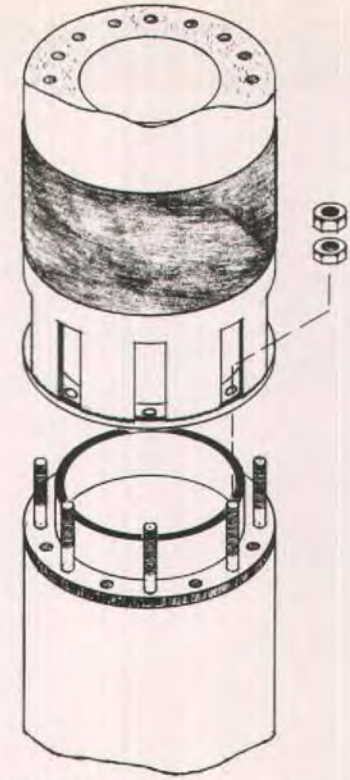


Fig. 40b. Schematic of bolted splice.

extremely important design factor.

The tractor trailer unit may not be a suitable vehicle to transport the longer poles in many areas which have narrow and winding roads. In such cases it may be worthwhile to consider segmental prestressed concrete poles.

SEGMENTAL PRESTRESSED CONCRETE POLES

Prestressed poles made of precast segments assembled at the site avoid problems associated with transporting the full length pole. These assemblies under full scale testing have revealed no adverse effect on the ultimate strength of the designed pole.

The assembly of segmental poles can and has been done in two ways. Either prestressed or nonstressed precast units are assembled to the required pole length and post-tensioned or precast prestressed pole segments can be spliced in one of four ways. In Japan (Fig. 39) a thick steel plate is tensioned to the ends of precast prestressed segments, and the pole is assembled by welding the segments together to the required length of the pole.

In Germany (Fig. 40), bolts are cast in the lower section of a precast prestressed segment. The upper segment has a galvanized steel section cast in with blockouts which accept the bolts that are double-nutted to make the connection. This area is covered with a galvanized steel cover plate.

Within the United States, two other splices have been developed and tested. The lap splice (Fig. 41) employs a galvanized steel section to be inserted into

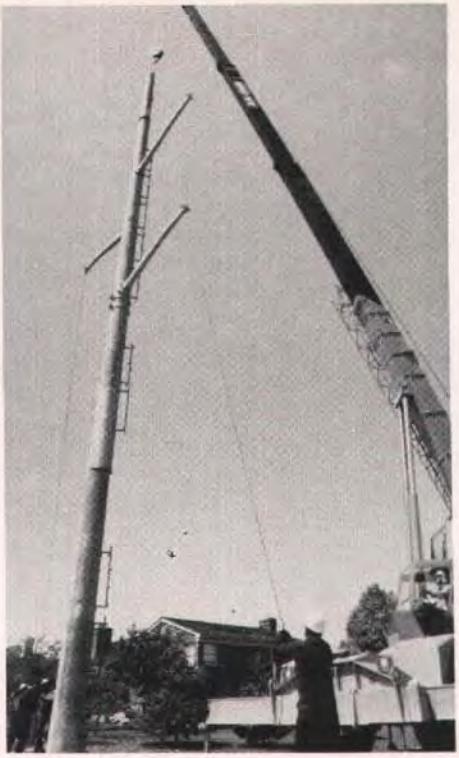


Fig. 41a. Slip joint splice.

the larger end of the form. The prestressing strand is pulled through and stressed to the form end plates. The concrete is cast into the small end of the tapered section. To develop the section's strength in the prestressing strand development areas, mild reinforcing steel is added. The larger end of the tapered steel section laps down over the smaller end of the segment below at least 1½ times the diameter of the pole sections at the splice. An air gap of 6 in. (152 mm) is designed into the splice between the plate in the splice unit and the top of the lower segment. A keyway is used to orient the segments correctly.

In the flange splice (Fig. 42), a steel plate is prestressed to the splice ends of the pole segments. The end plates are then bolted together like a pipe joint to develop the strength of the pole section.

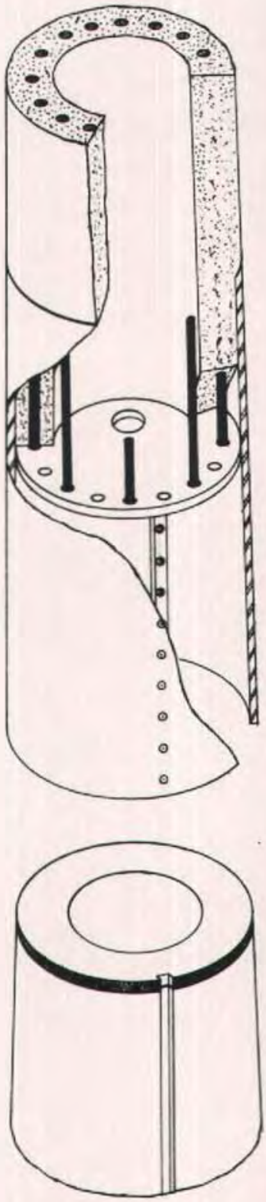


Fig. 41b. Schematic of slip joint splice.

Fig. 42a. Flange plate splice.

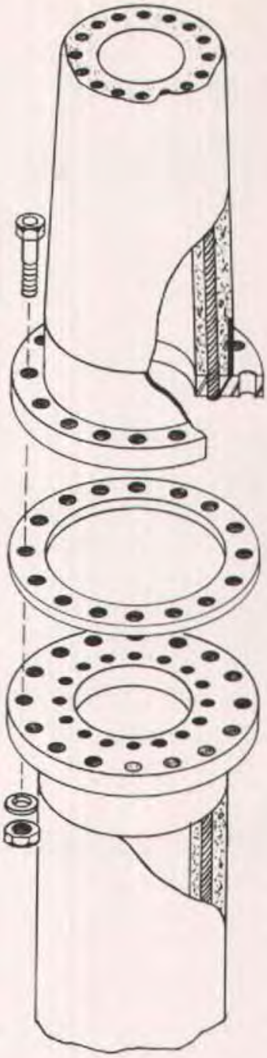


Fig. 42b. Schematic of flange plate splice.

FUTURE USE AND TECHNOLOGY

Most studies have indicated that the first cost of prestressed concrete poles is greater than timber, but less than steel, in the range of sizes of prestressed concrete poles designed and used today. It must be emphasized that numerous circumstances affect their economy, especially transportation and availability. From the data collected, the facts indicate that every country surveyed in the report is using prestressed concrete poles more and more frequently.

Future technology into higher strength concrete with a 28-day strength in the 10,000 to 12,000 psi (69 to 83 MPa) range may become standard as efforts to produce stronger poles with slender profile develop. Pole weights may also decrease with the use of lighter weight concrete. Sectionalized poles will reduce shipping and handling problems.

The development of concrete admixtures that provide concrete with acceptable electrical insulating and mechanical strength properties may open the way for innovative integrated design.

The increasing market for wood products may eventually eliminate wood from the pole marketplace; this will undoubtedly cause wood pole users to look at concrete poles as an attractive alternative.

Future technological developments in concrete pole design, manufacturing, and costing will have a very definite impact on the future use of prestressed concrete poles.

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* * *

NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by May 1, 1985.

COMPANY BROCHURES (AMERICAN-CANADIAN)

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5. Inficrete Poles, Barratt Spun Concrete Poles Ltd., P.O. Box 372, 4536 Montrose Road, Niagara Falls, Canada L2F 6T8.
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17. Spun Reinforced Concrete, Fratelli Carraro S.N.C., Vicenza, Italy.

* * *

APPENDIX
PRESTRESSED CONCRETE POLE APPLICATIONS

Fig. A1. Flag pole.

Fig. A2. Area lighting.

Fig. A3. Area lighting.

Fig. A4. Area lighting—Tennis court.

Fig. A5. Area lighting—Baseball field.

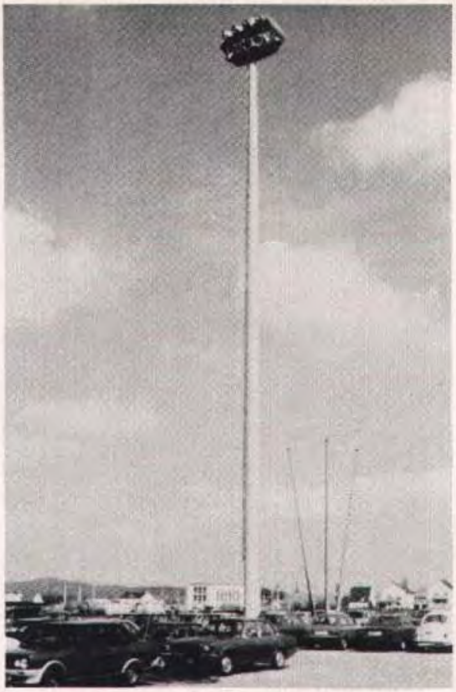

Fig. A6. High rise area lighting — Parking lot.

Fig. A7. High rise area lighting — Football field.

Fig. A8. Power distribution and street lighting.

Fig. A9. Power distribution and street lighting.

Fig. A10. Power distribution and street lighting.

Fig. A11. Power distribution and street lighting.

Fig. A12. Power transmission with distribution underbuild and area lighting.

Fig. A13. Power transmission with distribution underbuild.

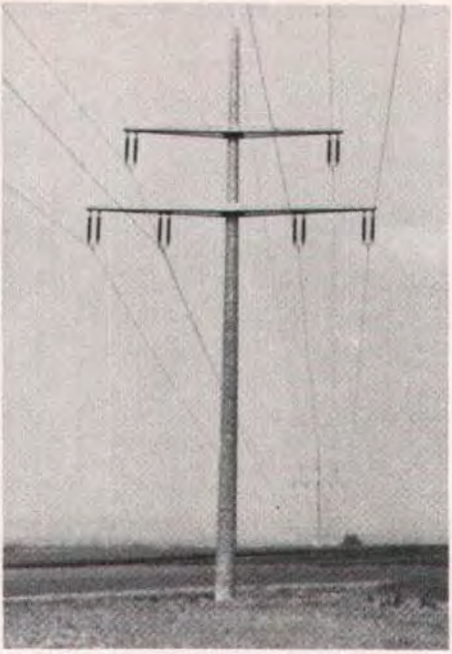

Fig. A14. European power transmission.

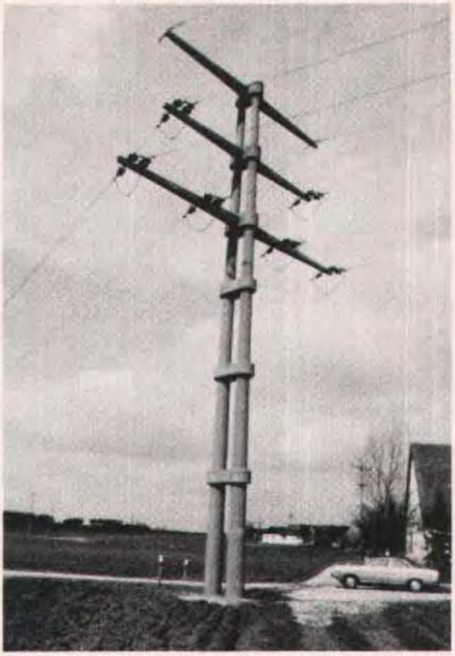

Fig. A15. European power transmission.

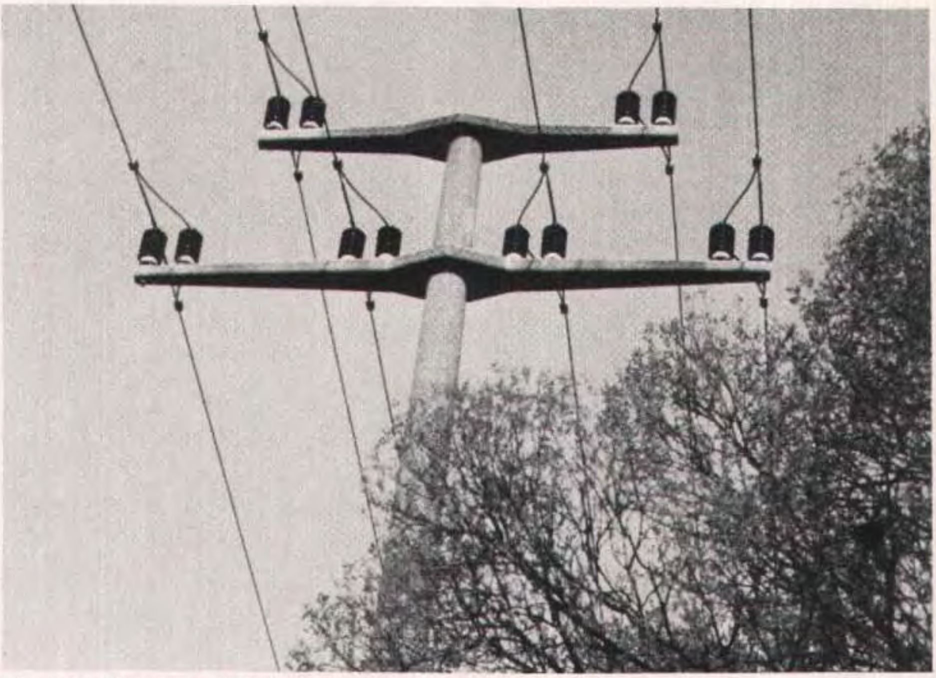

Fig. A16. Close-up of European power transmission.

Fig. A17. European power transmission.

Fig. A18. Power transmission.

Fig. A19. Power transmission.

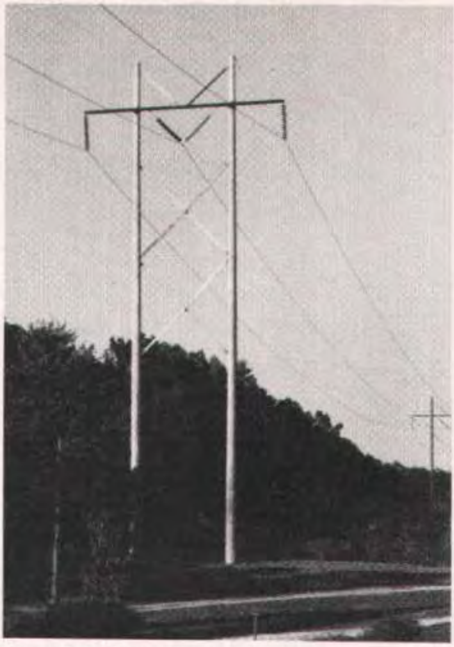

Fig. A20. Power transmission.

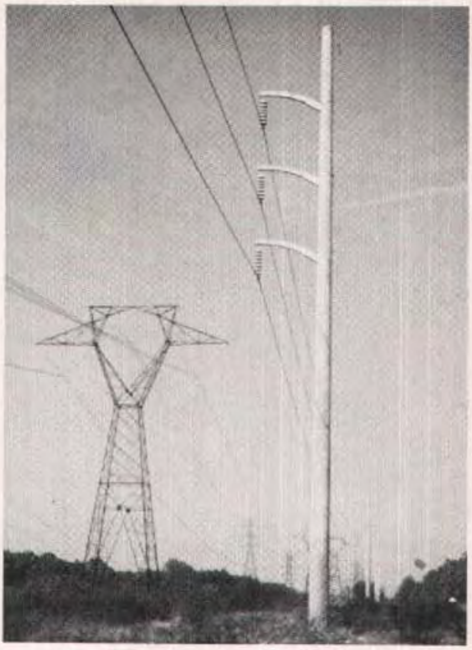

Fig. A21. Power transmission.

Fig. A22. Power transmission.

Fig. A23. Power transmission.

Fig. A24. Power transmission—Switch structures.

Fig. A25. Power transmission.

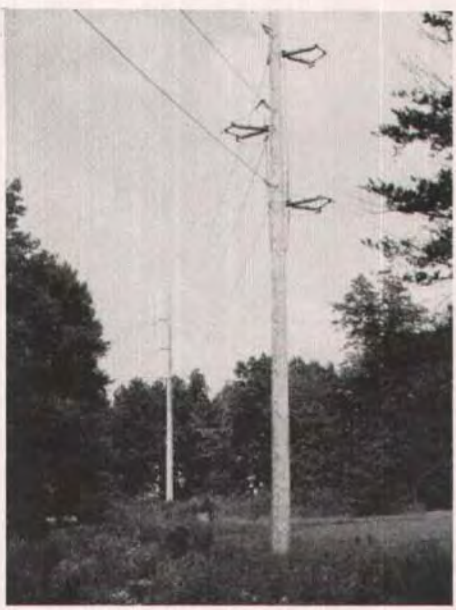

Fig. A26. Power transmission—Switch structures.

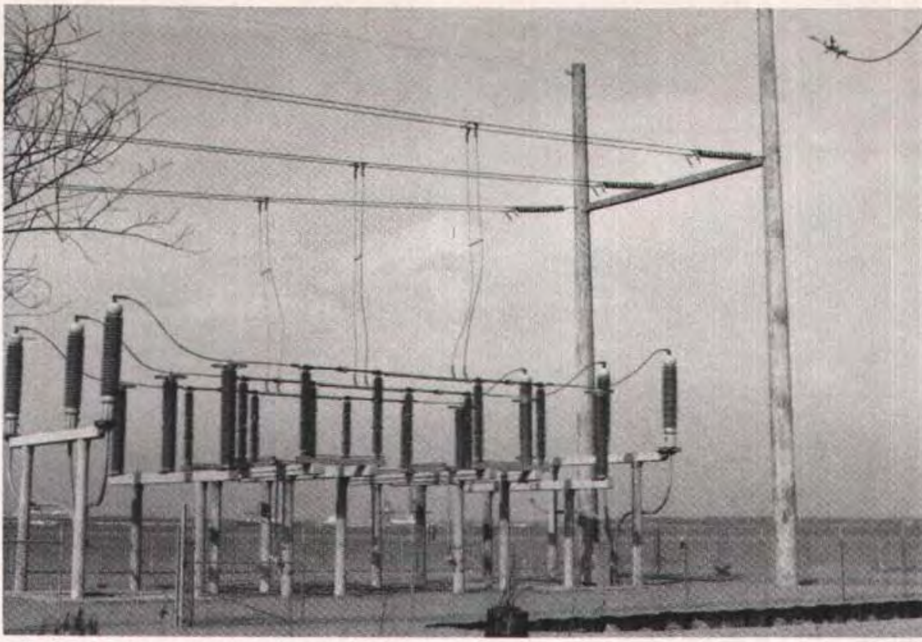

Fig. A27. Power overhead transmission to underground transmission.

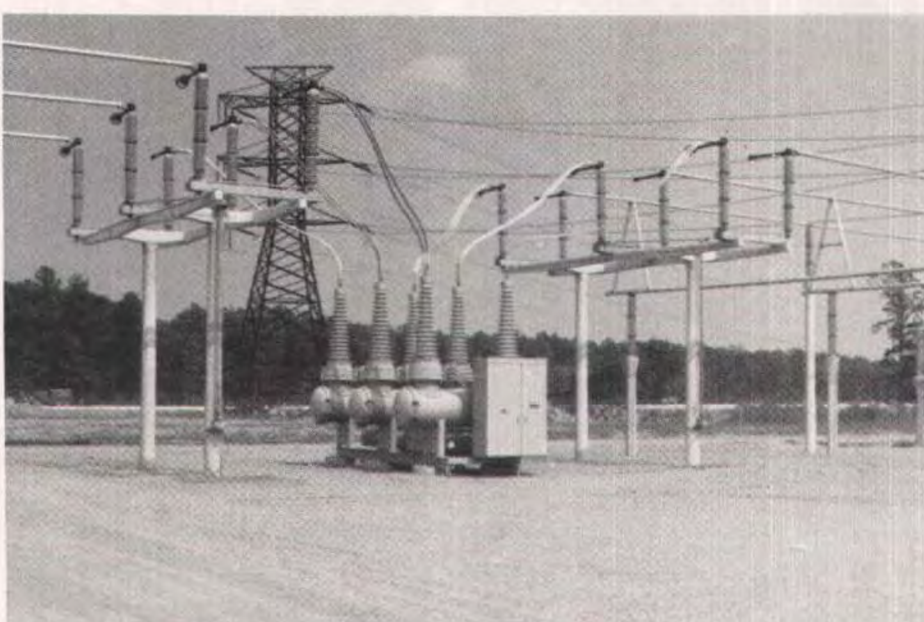

Fig. A28. Power substation structures.

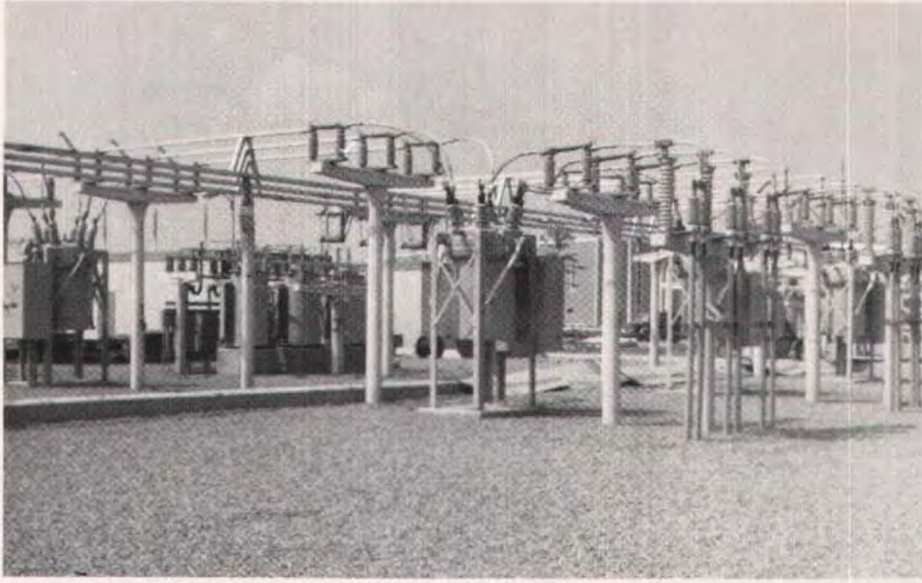

Fig. A29. Power substation structures.

Fig. A30. Power substation structures.

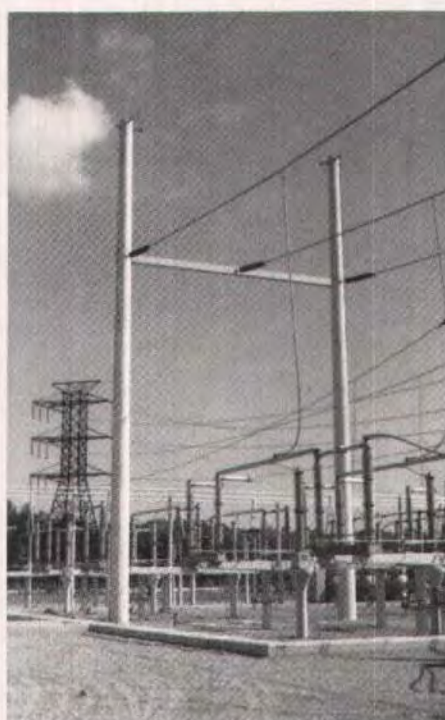

Fig. A31. Power substation structures.

Fig. A32. Communication structures — Microwaves and area lighting.

Fig. A33. Communication structures — Microwaves and area lighting.