

Australian/New Zealand Standard™

Structural design requirements for utility services poles



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This Joint Australian/New Zealand Standard was prepared by Joint Technical Committee CE/19, Utility Services Poles. It was approved on behalf of the Council of Standards Australia on 21 May 2000 and on behalf of the Council of Standards New Zealand on 16 May 2000. It was published on 30 August 2000.

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Electricity Engineers Association of New Zealand
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New Zealand Concrete Society
New Zealand Heavy Engineering Research Association
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Australian Aluminium Council
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Australian/New Zealand Standard™

Structural design requirements for utility services poles

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PREFACE

This Standard was prepared by the Joint Standards Australia/ Standards New Zealand Committee CE/19, Utility Services Poles.

The objective of this Standard is to provide power authorities, manufacturers, and designers, with the requirements for the design and installation of utility services poles.

This Standard is intended for use in conjunction with the following Standards:

AS

2209 Timber poles for overhead lines

AS/NZS

4065 Concrete utility services poles

4677 Steel utility services poles

The terms 'normative' and 'informative' have been used in this Standard to define the application of the appendix to which they apply. A 'normative' appendix is an integral part of a Standard, whereas an 'informative' appendix is only for information and guidance

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STANDARDS AUSTRALIA/STANDARDS NEW ZEALAND

Australian/New Zealand Standard Structural design requirements for utility services poles

SECTION 1 SCOPE AND GENERAL

1.1 SCOPE

This Standard sets out general requirements for structural design and minimum design loads applicable to pole structures supporting—

- (a) street or floodlighting;
- (b) road or railway signalling equipment;
- (c) aerial conductors carrying electric power, or communication signals;
- (d) equipment for communication through the atmosphere; or
- (e) any combination of these.

This Standard does not apply to lattice towers, guyed masts, masts or flag poles.

NOTES:

- 1 Design requirements for steel lattice towers and masts for communication purposes are given in AS 3995.
- 2 For information on the additional loads induced by the temporary attachment of flags or banners to utility services poles see Clause 3.8.

1.2 GENERAL

The structural design of utility services poles shall be based on accepted principles of structural mechanics taking due account of environmental and site factors for the particular locations in which the poles will be situated and their expected service life.

Design shall be either by calculation in accordance with Sections 2 to 5 inclusive, or by load testing in accordance with Section 7.

NOTE: A pole supplier should elect to use the calculation method for poles that are required in limited numbers, and to select the load-test method for poles that are required in sufficiently large numbers so that the expense of testing may be offset by potential material savings.

1.3 REFERENCED DOCUMENTS

The documents referred to in this Standard are listed in Appendix A.

1.4 DEFINITIONS

For the purpose of this Standard the definitions given in AS 1158.1, AS 1798 and those below apply.

1.4.1 Guyed/stayed

Stabilized above ground level against lateral forces by one or more steel cables, which are anchored at their lower ends to the ground or other permanent construction.

1.4.2 Guyed mast

A guyed lattice structure that relies on one or more sets of guys for its stability against multi-directional lateral forces.

1.4.3 Lattice towers/masts

Towers or masts in which the principal structural members are interconnected solely by triangulated bracing.

1.4.4 Pole

A freestanding structure that resists vertical loads as a column with or without bending, and resists lateral loads as a cantilever beam with or without torsion.

1.4.5 Guyed/stayed pole

A pole stabilized by one or more guys, for the purpose of resisting all, or part, of the resultant force from the applied forces.

1.4.6 Tower

A freestanding lattice structure, the lateral cross-sections of the structure generally being triangular, square or rectangular.

1.5 NOTATION

The quantity symbols used in this Standard shall have the meanings ascribed to them below.

The dimensional units for length, force and stress, in all expressions or equations in this Standard, shall be taken as metres (m), kilonewtons (kN) and megapascals (MPa) respectively. The dimensional units of stress raised to a fractional power shall be taken as megapascals.

Where non-dimensional ratios are involved, the numerator and denominator shall both be expressed in identical units.

A_b	= the contact face area of footing bearers (see Appendix I) (m ²)
A_f	= the area of one face of a banner or flag (see Appendix G) (m ²)
A_{fb}	= the minimum plan area of the underside of a footing (m ²)
b	= the effective width of an embedment (see Appendix I); or = the dimension of a banner or flag at right angles to the wind (see Appendix G) (m ²)
b_p	= the overall horizontal dimension of the pole cross-section perpendicular to the direction of the wind (m)
C	= $f_{bu} \cdot b$ (see Appendix I)
C_D, C_{df}	= wind-drag factors
D	= the overall depth of embedment of a pole or its footing (see Appendix I); or = the sag of an aerial cable or conductor (m)
d_c	= nominal overall diameter of an aerial cable (m)
d_{cs}	= the nominal overall diameter of an iced cable (m)
E	= Young's elastic modulus of elasticity
F_a	= aerial cable loads generally, or their action effects
F_{aG}	= the self-weight of a cable of length L_{eg} (kN)

F_{aR}	= the resultant total load on an aerial cable or conductor, which acts in the plane of the catenary (see Appendix F) (kN)
F_{aS}	= the dead load of snow and ice on a cable of length L_{eg} (kN)
F_{aT}	= the actions or action effects caused by cable temperatures T_s or T_{min} . (kN; kNm)
F_{aW}	= horizontal wind load on an aerial cable pole, due to wind perpendicular to the spans of the supported cables
F_{eq}	= earthquake loads generally, or their action effects (kN; kNm)
F_{gt}	= the sum of the vertical components of guy or stay tension (kN)
F_v	= the sum of the vertical forces on a pole other than F_{gt} (kN)
F_{wf}	= the total wind force on a banner or flag (see Appendix G) (kN)
f_b	= the permissible bearing strength of a foundation material at the serviceability limit state (kPa)
f_{bu}	= the permissible bearing strength of a foundation material at the strength limit state, assumed equal to $1.5 f_b$ (kPa)
f'_c	= the characteristic compressive strength of concrete after 28 days (MPa)
G	= dead loads generally, or their action effects (kN; kNm)
H_R	= the resultant of the horizontal forces (loads) acting on a pole (see Appendix I) (kN)
h_i	= the height difference
h_r	= the vertical height above ground level at which H_R acts (kN)
h_p	= the height of a pole above ground level (m)
k_I	= the wind resistance factor
K_T	= topographical factor (see Appendix E)
K_Z	= a terrain category/height factor (see Appendix E)
k_d	= the degradation factor (see Appendix D)
k_e	= a wind-gust factor
k_{wi}	= relative absorption of a test specimen after time 'j' (see Appendix H)
L_{eg}	= the effective span for dead-load (weight span) (see Figure 3.7.2) (m)
L_{ew}	= the effective (ruling) span for calculating cable tensions from wind-loads (m)
L_g	= the horizontal distance from the centre-line of a pole to the lowest point of the catenaries, formed on either side of it, of the aerial cable it supports (see Figure 3.7.2) (m)
L_i	= the horizontal length of the 'i'th span in a line of poles supporting aerial cables (m)
L_w	= the wind span (m)
L_1, L_2	= the horizontal distances from the pole being considered to the next pole upline and downline respectively (m)
M^*	= the design actions in bending, as determined from the relevant design load combination (kNm)
M_I	= structure importance multiplier
m_0, m_1, m_2, m_3	= mass of a test specimen at different times (see Appendix H) (g)

N^*	= the design action in tension or compression, as determined from the relevant design load combination (kN)
n	= the total number of spans between termination and/or deviation poles
p_b	= the basic regional wind pressure (see Appendix E) (kPa)
p_d	= the design horizontal wind pressure (see Appendix E) (kPa)
p_s	= the basic regional wind pressure for the serviceability limit state
p_u	= the basic regional wind pressure for the strength limit state
Q	= live loads and maintenance loads generally, or their action effects (kN; kNm)
q_z	= dynamic wind pressure at height z (kPa)
R	= nominal capacity (strength) of a member or part generally (bending, shear, torsion) (kN; kNm); or = a reaction force (kN)
R_u	= nominal capacity (strength) of a member or part at the strength limit state (kN; kNm)
S	= snow and ice loads generally, or their action effects (kN; kNm)
S_r	= the basic roof snow load (kPa)
T	= an aerial cable tension
T_s	= the local average daily minimum temperature taken over a 12 month period (°C)
$T_{min.}$	= a local average daily minimum temperature taken over a winter period (°C)
V^*	= the design action effect in shear, as determined from the relevant load combination (kN)
V_s	= basic regional wind speed at the serviceability limit state (m/s)
V_u	= basic regional wind speed at the strength limit state (m/s)
W	= wind loads generally, or their action effects (kN; kNm)
W_p	= wind loads generally, or their action effects, at the permissible stress limit state
W_s	= wind loads generally, or their action effects, at the serviceability limit state
W_u	= wind loads generally, or their action effects, at the strength limit state
w_{awt}	= component of the wind load causing tension in an aerial cable (kN/m)
w_g	= mass per metre length of an aerial cable (kg/m)
w_s	= unit weight of ice (kN/m ³)
w_w	= horizontal wind load per metre length of an aerial cable (kN/m)
z	= the height above ground of the attachment of a cable to a pole (m)
β	= horizontal deviation angle
θ	= angle between a vertical plane and the plane of the cable catenary when rotated by the wind
ϕ	= material capacity factor for particular action effects

1.6 CLASSIFICATION

1.6.1 General

Utility services poles shall be classified by—

- (a) intended usage in accordance with Clause 1.6.2;
- (b) consequence of collapse in accordance with Clause 1.6.3; and
- (c) where appropriate, vehicle impact resistance in accordance with Clause 1.6.4.

Poles in any class may be further qualified by the primary material from which they are constructed (e.g. timber, steel, concrete).

1.6.2 Usage classes

Usage classes of poles shall be distinguished as follows:

- (a) Street-lighting poles are poles intended to support luminaires that primarily illuminate public roads
- (b) Floodlighting poles are poles intended to support luminaires that primarily illuminate areas other than public roads
- (c) Aerial-conductor poles are poles intended to support
 - (i) electric power transmission and distribution conductors;
 - (ii) electric-traction conductors; or
 - (iii) communication conductors.
- (d) Traffic-signal poles are poles intended to support primarily road or railway signalling equipment
- (e) Communication poles are poles intended to support equipment that generates, relays, or receives communication signals through the atmosphere.
- (f) Multi-usage poles are poles intended for any combination of two or more of the above usages.

1.6.3 Importance classes

Importance classes shall be distinguished as follows:

- (a) Class I, where—
 - (i) the poles support vital post-disaster services; or
 - (ii) collapse of the poles and loss of the supported services would cause unacceptable danger to life, or extensive economic loss.
- (b) Class II, where—
 - (i) collapse of the poles would cause negligible danger to life and property; and
 - (ii) alternative arrangements can be provided if loss of the supported services occurs.
- (c) Class III, where collapse of the poles or loss of the supported services is more tolerable with respect to social and economic consequences than the above.

NOTE: Some guidance on the selection of importance classes is given in Appendix B.

1.6.4 Vehicle impact classes

Where utility services poles are located adjacent to the carriageway of a public road, their resistance to vehicle impact shall be classified as—

- (a) frangible, if they are intended to be deformed or displaced in a predictable and acceptable manner (see also Clause 2.6); or
- (b) rigid, if they are not intended to be frangible.

NOTE: Guidance on the selection of an appropriate vehicle impact class and the positioning of poles to minimize the effects of possible vehicle impact is given in Appendix B of AS/NZS 1158.1.3.

SECTION 2 DESIGN REQUIREMENTS AND PROCEDURES

2.1 GENERAL

2.1.1 Requirements

The design of a utility services pole and its component parts shall take into account, as appropriate, the limit states of stability, strength, serviceability and durability and any other relevant design condition in accordance with the requirements of this Section.

2.1.2 Procedure

The procedure for structural design shall be as follows:

- (a) The relevant loads and other actions shall be determined from Clauses 3.1 to 3.7 inclusive, and the design load for the particular limit state shall be determined from the appropriate combination in Clause 3.8.
- (b) The design action effect, S^* (bending moment, shear, torsion, deflection, rotation), shall be determined from the principles of structural mechanics, using the appropriate design loads.
- (c) The design capacity (ϕR) shall be determined either by calculation in accordance with Sections 4 and 5; or by load testing in accordance with Section 7 for the pole and with Section 6 for the foundations, where the capacity factor (ϕ) is taken as not greater than the appropriate value given in the relevant material design Standard, or in Section 6 for foundation materials.
- (d) The pole and its component parts shall be proportioned so that the design capacity, ϕR , is not less than the corresponding design action effect (S^*) i.e. $\phi R \geq S^*$.

2.2 DESIGN FOR STABILITY

Poles as a whole, including their embedment or footing as appropriate, shall be designed to prevent instability due to overturning, rotation and, if applicable, settlement, under the design load for stability determined from the relevant combination in Clause 3.9. The design resistance (strength) of the foundations shall be determined in accordance with Section 6.

2.3 DESIGN FOR STRENGTH

Poles and their component parts shall be designed for strength as determined by proportioning. At every cross-section, the design resistance (ϕR_u) is not less than the design action-effect (S^*) at the corresponding cross-section, where S^* is caused by the design loads for the strength limit state, determined from the relevant combination given in Clause 3.9.

2.4 DESIGN FOR SERVICEABILITY

Poles as a whole and their component parts, including their embedment or footings, shall be designed for serviceability. Deflections, rotations and, if appropriate, vibrations caused by the serviceability design loads determined from the relevant combination given in Clause 3.9, will be limited to values that will have negligible influence on the normal functioning and integrity of the supported service.

NOTE: Suggested serviceability limits are given in Appendix C.

2.5 DURABILITY

All types of utility services poles and their component parts, shall be designed to minimize loss of strength from any adverse effects of the environment on exposed surfaces, for the duration of their expected service life.

Unless otherwise specified herein, the requirement of this Clause shall be deemed to be satisfied if the durability requirements specified in the relevant material Standards are satisfied.

Alternatively, due allowance may be made in the design for a progressive degradation of strength over the duration of the expected service life.

NOTE: Guidance on nominal service lives for some applications in selected environments is given in Appendix D.

2.6 VEHICLE IMPACT

Poles that satisfy the requirements of Document 'NCHRP 350' shall be deemed to be frangible.

NOTE: Document NCHRP Report 350, Recommended procedures for the safety performance evaluation of highway features, National Cooperative Highway Research Program (Washington D.C.), 1993.

SECTION 3 LOADS AND LOAD COMBINATIONS

3.1 GENERAL

The design for stability, strength and serviceability of utility services poles and their component parts shall take account of the effects arising directly from the following loads, as appropriate:

- (a) Dead loads (G), determined from Clause 3.2.
- (b) Snow and ice loads (S), determined from Clause 3.3.
- (c) Wind loads (W), determined from Clause 3.4
- (d) Earthquake loads (F_{eq}), determined from Clause 3.5.
- (e) Live loads and maintenance loads (Q), determined from Clause 3.6.
- (f) Aerial cable loads (F_a), determined from Clause 3.7.

3.2 DEAD LOADS

Dead loads acting on a pole shall include the following, as appropriate for its intended use:

- (a) The self-weight of the pole and its component parts.
- (b) The weight of all attachments such as mounting and other brackets, luminaires, cables, transformers, signalling and communication equipment.

3.3 SNOW AND ICE LOADS

3.3.1 General

For poles in regions subject to snow, the snow load (S) shall be determined in accordance with AS 1170.3 or NZS 4203.1, as appropriate. The snow load shall be taken as the basic roof snow load (S_r) spread uniformly over the projected plan area of the pole and its attachments.

3.3.2 Australian conditions

For aerial conductor poles in regions of Australia subject to ice formation, the ice load on the conductors shall be calculated by assuming a radial thickness of surrounding ice, selected from Table 3.3(A) and a specific gravity of ice equal to 0.86.

TABLE 3.3(A)
ASSUMED THICKNESS OF ICE ON CONDUCTORS IN AUSTRALIA
(Unless local climatic conditions, topography and line directions are known
to cause more severe loads)

Region	Radial thickness of ice (m)
Alpine	$0.3d_c$
Sub-alpine	$0.2d_c$

3.3.3 New Zealand conditions

3.3.3.1 Zone classification

For aerial conductor poles in New Zealand three snow and ice zones should be considered, based broadly on latitude and NZS 4203. The zones for New Zealand are as follows:

- (a) *Zone N0* North Island—North. The parts of the North Island not covered in ice zone N1.
- (b) *Zone N1* North Island—South. The southern portion of the North Island, as defined by NZS 4203 snow zone 1 plus northern coastal areas of the South Island (e.g. Marlborough Sounds).
- (c) *Zone S* South Island. The whole of the South Island with the exception of northern coastal areas such as Marlborough Sounds, which may be considered as zone NI.

3.3.3.2 Loads on conductors

The ice load on conductors in New Zealand shall be calculated by assuming a radial thickness of ice, selected from Table 3.3(B) with ice of specific gravity of 0.7 used to represent the medium rime ice, likely to form in New Zealand. For conductors of 10 mm diameter or less, the radial ice thickness shall be increased by 10%. A coincident wind load shall be considered for the return period determined from Table 3.3(B).

The snow load on conductors in New Zealand shall be calculated by assuming a 30 mm radial thickness of 0.4 specific gravity wet snow with no coincident wind. Snow load shall be considered at altitudes above 500 m in Zone N0, 200 m in Zone N1, and at all altitudes in Zone S.

TABLE 3.3(B)
ICE AND WIND LOADS ON CONDUCTORS IN NEW ZEALAND

Zone	Altitude (m)	Ice thickness (mm)	Coincident wind return period (years)
N0 and N1	700–900	30	1
	900–1200	35	1
	1200–1500	40	5
S	500–700	30	1
	700–900	35	1
	900–1200	40	5
	1200–1500	45	5

3.4 WIND LOADS

Wind loads acting on a pole, its component parts and any attachments including aerial conductors fixed to it, shall be determined with either of the following;

- (a) In accordance with the appropriate wind loading Standard (AS 1170.2 or NZS 4203) as appropriate.
- (b) In accordance with Appendix E and the following:
 - (i) For the limit states of stability and strength, the wind load (W_u) shall be calculated using the basic wind pressure for ultimate strength (p_u).

- (ii) For the serviceability limit state, the wind load (W_s) shall be calculated using the basic wind pressure for serviceability (p_s).
- (iii) For aerial conductors subject to ice formation, the coexisting wind pressure (p_{si}) shall be taken as 0.7 kPa.
- (iv) For aerial conductors both the lateral wind load and the additional tension load induced by the wind shall be determined in accordance with Clause 3.7.

3.5 EARTHQUAKE LOADS

Where applicable and if significant, earthquake loads acting on poles and their attachments shall be taken into account.

Special pole structures supporting transformers, or similar heavy pole-top equipment, shall be designed to resist the earthquake loads determined from AS 1170.4 or NZS 4203, as appropriate.

NOTES:

- 1 In service, utility services poles have proven to not be susceptible to damage from ground accelerations due to earthquakes.
- 2 Loads induced in poles from displacements at fault lines or from foundation slumping cannot be rationally assessed. Some localized damage from these causes may occur.

3.6 LIVE LOADS AND MAINTENANCE LOADS

Where applicable, live loads and maintenance loads (Q) shall be considered to act concurrently on a pole and its component parts but, for the strength-limit state only, need not be taken to act concurrently with wind loads.

The live load for one person shall be taken as a concentrated vertical force of 1.4 kN, placed to cause the most adverse effect on the pole or the component part.

Each maintenance load shall be placed as a concentrated vertical force equal to the weight of the heaviest item of supported equipment, placed at the relevant point of its attachment to the pole.

3.7 AERIAL CABLE LOADS

3.7.1 General

Loads on poles and components parts, arising from aerial cables in the consecutive spans supported by the pole, shall be determined in accordance with Clauses 3.7.2 to 3.7.4 inclusive.

For poles in regions subject to snow or ice formation, the additional mass and size of the conductor, caused by ice formation, shall be determined in accordance with Clauses 3.3 and 3.4 respectively.

3.7.2 Dead loads

The dead load on a pole, arising from the support of aerial cables, shall be taken as the sum of self-weight of the cables and, where appropriate, the weight of suspension attachments (insulators and the like), the weight of snow or ice on the cables and the vertical component of any stay tensions, calculated separately, as follows:

- (a) The self-weight of the cable (F_{aG}) shall be calculated from the following equation:

$$F_{aG} = w_g \times 9.8 \times 10^{-3} \times L_{eg} \text{ (kN)} \quad \dots 3.7(1)$$

where

w_g = the mass per metre length of cable (kg/m)

L_{eg} = the effective span (weight span) in metres, taken as the sum of the horizontal distances from the centre-line of the pole to the lowest point of the catenaries formed on either side of the pole (see Figure 3.7.2)

NOTE: If the attachment points of the cables, to the three poles supporting adjoining spans, are at the same level, L_{eg} is the mean value of the spans on either side of the pole being considered.

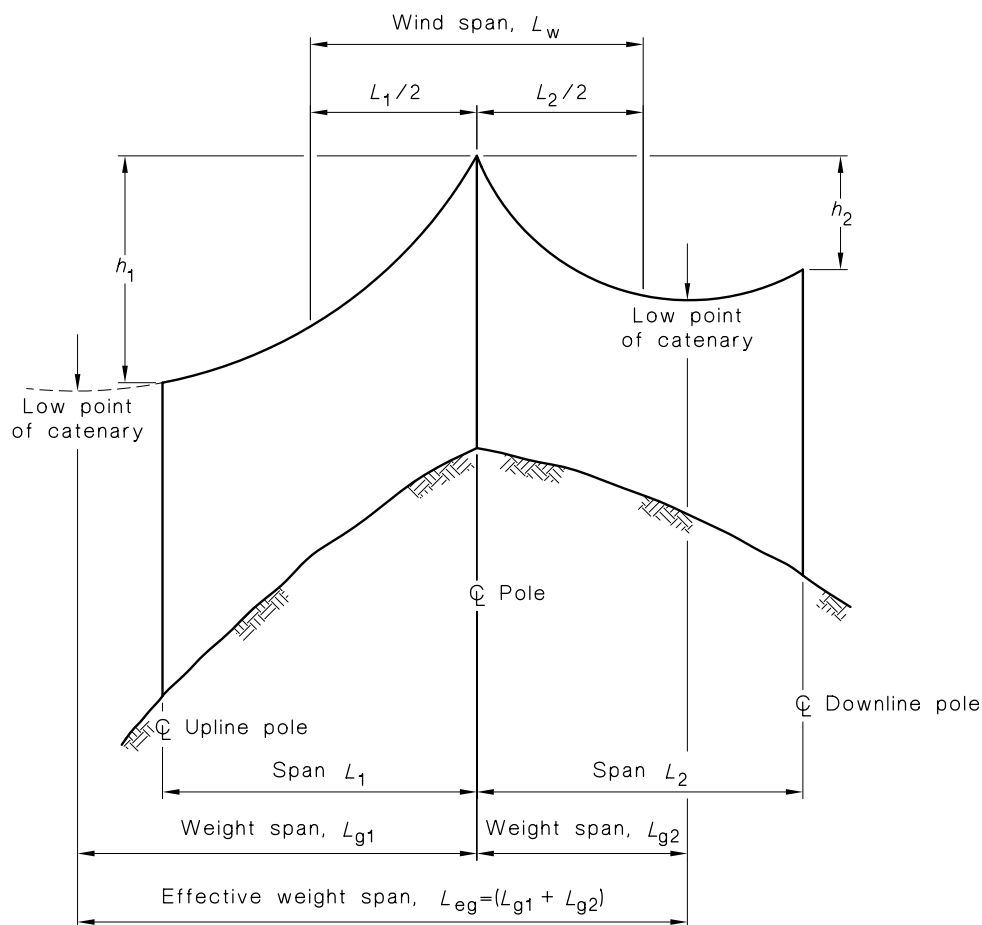


FIGURE 3.7.2 EFFECTIVE SPANS

- (b) In regions subject to the formation of ice, the weight of ice (F_{as}) shall be calculated from the following equation:

$$F_{as} = \frac{w_s}{4} [(d_{cs}^2 - d_c^2)] L_{eg} \text{ (kN)} \quad \dots 3.7(2)$$

where

- w_s = unit weight of ice (kN/m^3)
- d_{cs} = nominal overall diameter of the iced cable in metres determined from Clause 3.3
- d_c = nominal external diameter of the cable in metres
- L_{eg} = the effective span, determined as for Item (a) above

- (c) The vertical component of a stay tension is calculated as the product of the tension (in kilonewtons) and the cosine of the angle formed between the stay and a vertical axis through its point of attachment to the pole.

3.7.3 Wind load

The horizontal load on a pole from supported aerial cables (F_{aw}), arising from wind at right angles to the span of the cables, shall be calculated from the following equation:

$$F_{aw} = p_d \cdot d_c \cdot k_e \cdot L_w \text{ (kN)} \quad \dots 3.7(3)$$

where

- p_d = the design wind pressure in kilopascals, determined in accordance with Appendix E for the relevant limit state
- d_c = nominal external diameter of the cable, in metres including the increase due to snow or ice where applicable
- k_e = a span factor, taken as equal to 1.0 for the serviceability limit state, or determined from Figure 3.7.3 for the strength limit state as appropriate
- L_w = the wind span
= $(L_1 + L_2)/2$ (see Figure 3.7.2)

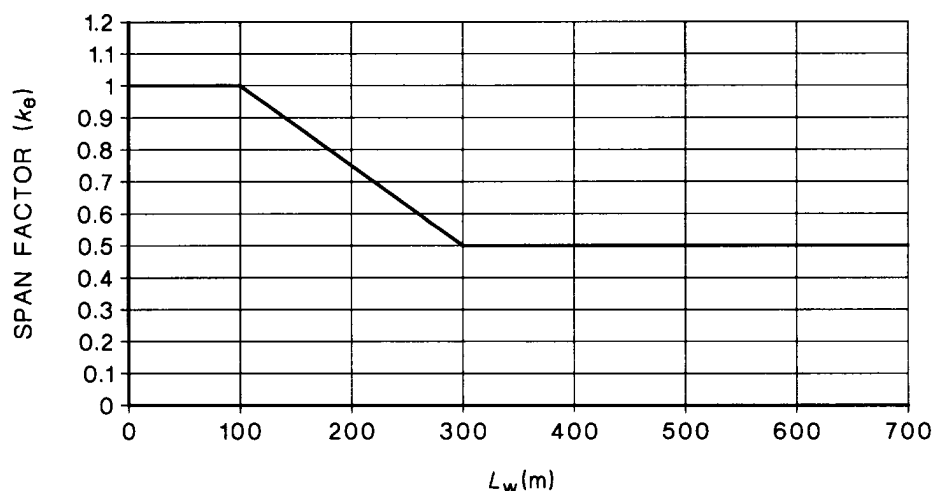


FIGURE 3.7.3 SPAN FACTOR (k_e) FOR STRENGTH LIMIT STATE

3.7.4 Aerial cable tension loads

3.7.4.1 General

In a line of poles, the effects on the strength and stability of a pole arising from cable tensions shall be assessed and appropriate values included in the design load combinations.

The assessment shall be made at poles where a line of aerial cables—

- (a) terminates; or
- (b) changes direction by a horizontal deviation angle.

NOTES:

- 1 In general there is sufficient flexibility in the poles, cross-arms and attachments to produce a uniform tension in all spans of cable line under a particular set of environmental conditions (temperature, wind, ice).
- 2 Details of sag/tension calculations are given in Appendix F
- 3 Where large differentials in span lengths occur in a series of spans, consideration should be given to the differential along-line tensions that may arise from this cause.

3.7.4.2 Cable tensions

NOTE: For the purpose of Clause 3.7.4.1, cable tensions may be calculated in accordance with Appendix F.

For the purpose of calculating the line tension, due to the wind-load component F_{aw} (at a given temperature) from Clause 3.7.3, L_w shall be taken as the effective (ruling) span (L_{ew}) determined from the following equations:

$$(a) \quad L_{ew} = \sqrt{\frac{\sum_{i=1}^{i=n} (L_i)^3}{\sum_{i=1}^{i=n} I_i}} \quad \text{for level spans; or} \quad \dots 3.7(4)$$

$$(b) \quad L_{ew} = \sqrt{\frac{\sum_{i=1}^{i=n} (L_i^4 / I_i)}{\sum_{i=1}^{i=n} I_i}} \quad \text{for inclined spans} \quad \dots 3.7(5)$$

where

$$I_i = \sqrt{L_i^2 + h_i^2}$$

L_i = the horizontal length of the 'ith' span
 n = the total number of spans between termination or deviation poles, or both
 h_i = the height difference between the conductor supports at each end of the 'ith' span

These equations apply only if no span in the group of n spans is outside the range $L_{ew}/2$ to $2L_{ew}$.

3.7.4.3 Tensions due to temperature changes

Changes in cable tensions due to temperature variations (F_{aT}) shall be taken as the tensions arising from a cable temperature equal to—

- (a) the 12 month mean of the local daily minimum temperatures, for load combinations with wind load;
- (b) the mean of the local winter season daily minimum temperatures, for load combinations without wind loads; or
- (c) a temperature of 0°C coexisting with snow and ice loading.

NOTE: The options above should take account of the cooling effects due to wind velocity. The temperatures referred to may be obtained from the relevant Meteorological Authority.

3.7.5 Stay loads

In a line of poles, at a pole where the aerial cables either terminate or deviate from a continuous straight line, or where the strength or stability of the pole, or both, is insufficient to resist the horizontal resultant of the cable tensions, the pole may be strengthened or stabilized, or both, by means of stays or braces.

The stays or braces shall be designed, in accordance with the appropriate material Standard, to resist the relevant forces arising from the actions of the strength limit state loads on the cables. Their anchorage, at the ends remote from the pole, shall be capable of developing resistance to the consequent pull-out, uplift and sliding forces, assuming an appropriate capacity factor for each type of resistance but not more than 0.9.

Consideration shall be given to the effects of redistribution of forces between the guy and rigid based poles under various load combinations.

3.7.6 Torsional loads

Poles with high torsional stiffness, such as hollow tubular or solid square or circular sections, shall be designed to resist torsion loads arising from broken conductors, cross-arms or insulators, unless some other means is provided to protect the pole from torsional failure (see Note 1).

Poles with low torsional stiffness but high torsional resilience, and low along-line stiffness, need not be designed for torsional loading arising from conductor, cross-arm or insulator failures (see Note 2)

NOTES:

- 1 Some of the effective means of protecting torsionally stiff poles from twisting forces are allowing rotation in the ground, limiting the cross-arm strength or stiffness, providing shear pins, swinging insulators or slipping bindings between the conductor and the insulator.
- 2 Steel or prestressed concrete *I*, *C* or truss-section poles having a minimum down-line strength of 25% of their transverse strength can be expected to twist and bend without significant damage until the out-of-balance loads are equalized.
- 3 Other more stringent requirements may be required by the relevant authority in particular circumstances such as for lines crossing railway tracks.
- 4 Terminal poles with a reduced strength transverse to the line should be assessed for the effects of bi-axial bending.

3.8 LOADS ON FLAGS AND BANNERS

Where a pole has provision for the attachment of flags or banners, allowance shall be made also for the additional forces due to wind action on a specified maximum size of flag or banner.

NOTE: Guidance on the determination for forces arising from wind action on flags or banners is given in Appendix G.

3.9 DESIGN LOAD COMBINATIONS

3.9.1 Design load

The design loads on a pole, for a particular limit state, shall be taken as the combination of factored loads producing the most adverse effect on the pole, where such combinations are determined from, but not limited to, the combinations given in Clauses 3.9.2 to 3.9.4. For regions *not* subject to snow or ice, *S* is to be taken as zero and F_{aT} only applies to poles supporting aerial cables.

3.9.2 Combinations for the strength limit state

As a minimum, the following combinations shall be considered for the strength limit state:

- (a) $1.1 G + 1.5 Q + 1.2 S + 1.2 F_{aT}$

- (b) $1.1 G + k_l W_u + S + F_{aT}$
- (c) $1.1 G + F_{eq} + F_{aT}$
- (d) $0.9 G + k_l W_u + F_{aT}$

The structure importance multiplier (M_i) is obtained from Table 3.9 and the importance classes are defined in Clause 1.6.3.

TABLE 3.9
STRUCTURE IMPORTANCE MULTIPLIER (M_i)

Importance class	Factor M_i
I	1.00
II	0.85
III	0.75

3.9.3 Combinations for the stability limit state

The combinations for the stability limit state shall be determined as follows:

- (a) Each type of load shall be subdivided into components tending to cause instability and components tending to resist instability.
- (b) The components tending to cause instability shall be factored in accordance with Clause 3.9.2.
- (c) The components tending to resist instability shall each be multiplied by the common factor 0.8.
- (d) The combinations for the stability limit state shall be obtained from those given in Clause 3.9.2 by substituting for each type of load therein the value obtained from Item (b) above reduced by the value obtained from Item (c) above. The importance factor (k_l) shall be applied as for Clause 3.9.2.

3.9.4 Combinations for the serviceability limit state

The following combinations shall be considered for the serviceability limit state:

- (a) $G + Q + S + F_{aT}$
- (b) $G + Q + W_s + F_{aT}$
- (c) $G + W_s + 0.85(S + F_{aT})$

For aerial conductors in regions subject to ice formation, W_s in combination (c) shall be determined in accordance with Clause 3.4(b)(iii).

SECTION 4 DESIGN REQUIREMENTS FOR MATERIALS

4.1 STEEL

4.1.1 General

Design properties for structural steel materials and steel fasteners shall be as given in AS 4100, NZS 3404.1 or AS/NZS 4600, as appropriate, and other applicable Standards, unless otherwise specified in this Section.

4.1.2 Minimum thickness

The thickness of steel plate used in any structural pole elements shall be not less than 1.6 mm.

4.1.3 Requirements for plate thickness less than 3 mm

Where the thickness of steel plate used in a pole is less than 3 mm, the following requirements apply:

- (a) **Welding** Special attention shall be given to weld quality in thin-walled elements and in particular to the avoidance of weld undercut.
- (b) **Fatigue** Structural detailing shall avoid stress concentrations and connections subject to cyclic loading which rely on the localized bending resistance of thin-walled components.
- (c) **Handling** Consideration should be given to the need for special handling of thin-walled elements to avoid localized distortion.
- (d) **Durability** Due consideration should be given to the potential for accelerated corrosion at and below ground level.

4.1.4 Serviceability

4.1.4.1 General

Steel poles shall meet the serviceability criteria, appropriate to the use of the pole, set out in Clauses 4.1.4.2 to 4.1.4.3.

4.1.4.2 Deflection and rotation

For all types of poles, deflection and rotation parameters shall be determined by the operating system constraints.

4.1.4.3 Durability

Where appropriate, steel poles shall be given a protective treatment consistent with their intended service life (see Note 1).

In some situations, the methods for achieving required design life for ground-planted poles may include any of the following:

- (a) Applying a heavier galvanized coating to the embedded section of the pole.
- (b) Increasing the thickness of the embedded steel section (see Note 2).
- (c) Applying a protective membrane or paint coating.
- (d) Embedding in concrete backfill.
- (e) Any other appropriate action.

NOTES:

- 1 Guidance on the protection of steel against atmospheric corrosion is given in AS/NZS 2312.
- 2 A relatively small corrosion allowance in the embedded section of the pole may increase service life significantly, e.g. 1 mm increase in steel thickness in normal soil can increase pole life by 10–20 years or more.

4.1.5 Low temperature requirements

Where applicable, steel grades for poles subject to low temperature conditions shall be chosen in accordance with the requirements for brittle fracture resistance given in AS 4100 or NZS 3404.1 as appropriate.

4.2 CONCRETE**4.2.1 General**

Design properties for concrete, reinforcement and tendons shall be as given in AS 3600 or NZS 3101.1, or as specified in Clauses 4.2.2 to 4.2.6.

4.2.2 Strength**4.2.2.1 Characteristic or specified compressive strength**

The characteristic or specified compressive strength at 28 days (f'_c), shall be not less than 40 MPa.

4.2.2.2 Characteristic flexural tensile strength or modulus of rupture

The characteristic flexural tensile strength or modulus of rupture after 28 days of standard curing may be taken as one of the following values as appropriate:

- (a) For pole elements subject to sustained tensile stresses, $0.6\sqrt{f'_c}$.
- (b) For pole elements subject to transient tensile stresses, $0.8\sqrt{f'_c}$.

4.2.3 Strength capacity factor

For poles designed by load testing in accordance with Section 7, the strength capacity factor (ϕ) shall be taken as not greater than 1.0.

For poles designed by calculation, ϕ shall be taken as not greater than the following values, as appropriate for the type of action effect being considered:

- (a) Bending, 0.9.
- (b) Compression, shear, or torsion, or any of these in combination, 0.8.
- (c) Bearing, 0.7.

4.2.4 Serviceability**4.2.4.1 General**

Concrete poles shall meet the serviceability criteria, appropriate to the use of the pole, set out in Clauses 4.2.4.2 and 4.2.4.3.

4.2.4.2 Deflection and rotation

For electromotive transport poles, communication equipment poles, and some floodlighting poles, deflection and rotation parameter shall be determined by the operating system constraints. For most other uses, deflection and rotation shall not be considered a serviceability constraint unless specified by the purchaser.

4.2.4.3 Crack width

Crack widths at the serviceability limit state shall not exceed 0.25 mm unless otherwise specified. For sustained dead loads or cable tension loads, the long-term effects of creep and shrinkage shall be considered.

NOTE: For further information on concrete crack width see Appendix D.

4.2.5 Concrete cover

4.2.5.1 Exposure classifications

For the purposes of Clauses 4.2.5.2 and 4.2.5.3, the exposure classification for poles shall be determined in accordance with AS 3600 Section 4, or NZS 3101.1 Section 5, as appropriate.

4.2.5.2 Exposure classifications other than C, or U more severe than C

For all exposure classification other than C, or other than U more severe than C, the clear cover to reinforcement (including tie wires) and tendons shall be not less than the greatest of —

- (a) the maximum nominal aggregate size;
- (b) three-quarters of the nominal diameter of the bar, wire or tendon to which the cover is measured; or
- (c) when tested in accordance with Appendix H, if —
 - (i) absorption $\leq 5.5\%$, cover = 9 mm;
 - (ii) $5.5\% < \text{absorption} \leq 6.5\%$, cover = 19 mm; or
 - (iii) absorption $> 6.5\%$, cover as per AS 3600/NZS 3101.1; or
- (d) other methods of providing suitable durability

4.2.5.3 Exposure classification C, or U more severe than C

For exposure classification C, or U more severe than C, or for poles within 1 km from a coastline with prevailing onshore winds, in addition to the requirements of Clause 4.2.5.2 one or more of the following protective actions should be adopted to achieve the required design life:

- (a) Increase the thickness of concrete cover.
- (b) Increase the specified strength grade, or otherwise reduce the permeability of the concrete.
- (c) Apply a protective coating to exposed surfaces.
- (d) Apply a corrosion-resistant coating to the reinforcement or tendons.
- (e) Provide cathodic protection to the reinforcement or tendons.
- (f) Seal the base of spun concrete poles.
- (g) Any other appropriate action.

4.2.6 Reinforcement and tendons

4.2.6.1 General

All reinforcement and tendons shall be effectively maintained in their correct position during manufacture of the pole. All supports used for this purpose shall be made from durable and stable materials that are not deleterious to the concrete or the reinforcement.

4.2.6.2 Poles designed by load testing

For poles designed by load testing in accordance with Section 7, the following exceptions apply to the requirements for reinforcement and tendons specified in AS 3600 or NZS 3101.1:

- (a) The minimum clear distances between parallel bars and tendons may be waived.
- (b) Lateral restraint of compression reinforcement by ties, or similar fitments, may be omitted.
- (c) Enclosure of bundled bars, or bundled tendons, within ties or similar fitments may be omitted.
- (d) Shear reinforcement may be omitted if the tested prototypes contain no shear reinforcement and the tests demonstrate that the design strength can be achieved without failure.

4.2.6.3 Poles designed by calculation

For poles designed by calculation, shear reinforcement may be omitted if the calculated shear strength provided by the concrete alone is not less than the minimum levels specified in AS 3600 or NZS 3101.1 for the omission of shear reinforcement in beams.

4.3 TIMBER

4.3.1 General

Design properties and design methods for timber utility poles and components shall be in accordance with AS 1720.1 or NZS 3606. Where specifically defined for round timbers, they shall be in accordance with Clauses 4.3.2 to 4.3.4.

4.3.2 Characteristic strengths and elastic moduli

The characteristic strengths and elastic moduli for untrimmed poles that conform in quality to the grade requirements specified in AS 2209 shall be as specified in Tables 4.3.1 and 4.3.2, unless verified by ingrade testing.

Strength groups and joint group classifications shall be assigned to species in accordance with AS 1720.2.

TABLE 4.3.1

POLE TIMBERS GRADED TO AS 2209 — RELATIONSHIP BETWEEN STRENGTH GROUPS AND CHARACTERISTIC PROPERTIES (MPa)

Strength group	Stress grade	Bending (f'_b) ⁽³⁾	Tension parallel to grain (f'_t) ⁽³⁾		Shear (f'_s) ⁽³⁾	Compression parallel to grain (f'_c) ⁽³⁾	Short duration modulus-of-elasticity (E)
			Hardwood	Softwood			
S1	F34	100	60	—	7.2	75	21500
S2	F27	80	50	—	6.1	60	18500
S3	F22	65	40	—	5.0	50	16000
S4	F17	50	30	26	4.3	40	14000
S5	F14	40	25	21	3.7	30	12000
S6	F11	35	20	17	3.1	25	10500
S7	F8	25	15	13	2.5	20	9100

NOTES:

- 1 The equivalence expressed in the table above is based upon the assumption that the poles are cut from mature trees.
- 2 The modulus of elasticity includes an allowance of about 5% for shear deformation.
- 3 See Clause 4.3.4.1.

TABLE 4.3.2
CHARACTERISTIC STRENGTH PROPERTIES (MPa) FOR BEARING AND SHEAR AT JOINTS

Strength group	Bearing		Shear at joint details (f'_s) (see Note)
	Perpendicular to grain (f'_n) (see Note)	Parallel to grain (f'_t) (see Note)	
S1	60	—	7.2
S2	50	—	6.1
S3	40	—	5.0
S4	30	26	4.3
S5	25	21	3.7
S6	20	17	3.1
S7	15	13	2.5

NOTE: See Clause 4.3.4.1.

4.3.3 Design factors—Material

4.3.3.1 Capacity factor

Values for the capacity factor (ϕ), for calculating the design capacity of poles (ϕR), shall be determined using Table 4.3.3.

TABLE 4.3.3
CAPACITY FACTORS FOR UTILITY POLES

Basis for determining characteristic strength properties	Characteristic design property to which the value of ϕ shall apply for calculating the design capacity	ϕ
Poles graded to AS 2209	All properties	0.90
Poles graded using Proof Grading in accordance with Section 7 of draft code	f'_b (see Note) All other properties	0.95 0.90
Poles with bending properties established from in grade evaluation and subject to periodic testing/monitoring of properties	f'_b (see Note) All other properties	0.95 0.90

NOTE: See Clause 4.3.4.1.

4.3.3.2 Duration of load effects (strength)

The effect of duration of load on strength of timber poles and components is given by the modification factor k_1 , as defined in Table 4.3.4. The effective duration of load refers to the cumulative duration for which the peak load occurs. Guidelines for determination of the effective duration of load are detailed in AS 1720.1.

4.3.3.3 Duration of load effects (stiffness)

For timber poles subject to sustained bending, creep effects should be considered. The effect of duration of load on stiffness of timber poles and components shall be determined in accordance with AS 1720.1 or NZS 3603. For other timber components, the short-term deflection shall be multiplied by the appropriate creep factor j_2 or j_3 , as given in AS 1720.1 or NZS 3603

TABLE 4.3.4
DURATION OF LOAD FACTOR FOR STRENGTH

Type of load	Effective duration of peak load	Modification factor (k_1) for strength of poles and timber components (see Note)	Modification factor (k_1) (see Note) for strength of timber connections using laterally loaded fasteners
Instantaneous (e.g. ultimate wind and earthquake)	5 seconds	1.00	1.14
Short term (e.g. construction maintenance)	5 hours	0.97	0.86
Medium term (e.g. snow/ice in sub-alpine areas)	5 days	0.94	0.77
Long term (e.g. snow/ice in alpine areas)	5 months	0.80	0.69
Permanent	> 1 years	0.57	0.57

NOTE: See Clause 4.3.4.1.

4.3.3.4 Pole degradation factors

For all timber poles, the design shall allow for loss of strength and stiffness associated with degradation of the critical section of the pole at and below the ground line over its expected design life. The degradation factor (k_d) shall be determined from Table 4.3.5, unless more accurate durability information is available.

The values of k_d given in Table 4.3.5 are based upon expected loss of effective section. In cases where the local environment in which the pole will be located is known to be of high hazard (e.g. due to excessive moisture or high probability of insect attack) more conservative values may be appropriate.

NOTE: Where a systematic inspection and maintenance program (see Section 4.3.5) is in place and where any evidence of degradation is effectively preservative treated (e.g. using diffusion preservatives), then the values of k_d given in Table 4.3.5 for untreated timbers will be conservative and higher values may be appropriate.

TABLE 4.3.5
POLE DEGRADATION FACTORS

Type of pole	Design life (years)	Pole diameter $d < 250$ mm k_d	Pole diameter $250 \leq d \leq 400$ k_d	Pole diameter $d > 400$ mm k_d
Full length preservative-treated softwood in accordance with AS 2209	20	1.0	1.0	1.0
	50	0.80	0.85	0.90
Full length preservative-treated hardwood in accordance with AS 2209	20	1.0	1.0	1.0
	50	0.80	0.85	0.90
Durability Class 1 untreated hardwood in accordance with AS 2209	20	0.80	0.90	1.0
	50	0.50	0.55	0.60
Durability Class 2 untreated hardwood in accordance with AS 2209	20	0.70	0.80	0.90
	50	0.30	0.40	0.45