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Overhead transmission lines – Design criteria

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colour inside

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OVERHEAD TRANSMISSION LINES – DESIGN CRITERIA

FOREWORD

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International Standard IEC 60826 has been prepared by IEC technical committee 11: Overhead lines.

This fourth edition cancels and replaces the third edition published in 2003. It constitutes a technical revision.

The main technical changes with regard to the previous edition are as follows:

This standard has been further simplified by removing many informative annexes and theoretical details that can now be found in CIGRE Technical Brochure 178 and referred to as needed in the text of the standard. Many revisions have also been made that reflect the users experience in the application of this standard, together with information about amplification of wind speed due to escarpments. The annexes dealing with icing data have also been updated using new work by CIGRE.

The text of this standard is based on the following documents:

FDIS	Report on voting
11/251/FDIS	11/252/RVD

Full information on the voting for the approval of this International Standard can be found in the report on voting indicated in the above table.

This document has been drafted in accordance with the ISO/IEC Directives, Part 2.

The committee has decided that the contents of this document will remain unchanged until the stability date indicated on the IEC website under "http://webstore.iec.ch" in the data related to the specific document. At this date, the document will be

- reconfirmed,
- withdrawn,
- replaced by a revised edition, or
- amended.

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OVERHEAD TRANSMISSION LINES – DESIGN CRITERIA

1 Scope

This International Standard specifies the loading and strength requirements of overhead lines derived from reliability-based design principles. These requirements apply to lines 45 kV and above, but can also be applied to lines with a lower nominal voltage.

This document also provides a framework for the preparation of national standards dealing with overhead transmission lines, using reliability concepts and employing probabilistic or semi-probabilistic methods. These national standards will need to establish the local climatic data for the use and application of this standard, in addition to other data that are country-specific.

Although the design criteria in this standard apply to new lines, many concepts can be used to address the design and reliability requirements for refurbishment, upgrading and uprating of existing lines.

This document does not cover the detailed design of line components such as supports, foundations, conductors or insulators strings.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

IEC 60652, *Loading tests on overhead line structures*

IEC 61089, *Round wire concentric lay overhead electrical stranded conductors*

IEC 61773, *Overhead lines – Testing of foundations for structures*

IEC 61774, *Overhead lines – Meteorological data for assessing climatic loads*

IEC 61284, *Overhead lines – Requirements and tests for fittings*

3 Terms, definitions, symbols and abbreviations

For the purposes of this document, the following terms, definitions, symbols and abbreviations apply.

3.1 Terms and definitions

3.1.1
characteristic strength
guaranteed strength, minimum strength, minimum failing load
R_c
strength value guaranteed in appropriate standards

Note 1 to entry: This value usually corresponds to an exclusion limit, from 2 % to 5 %, with 10 % being an upper practical (and conservative) limit.

3.1.2
coefficient of variation
COV
ratio of the standard deviation to the mean value

Note 1 to entry: The *COV* of load and strength are respectively denoted by v_Q and v_R.

3.1.3
components
different parts of a transmission line system having a specified purpose

Note 1 to entry: Typical components are supports, foundations, conductors and insulator strings.

3.1.4
damage limit (of a component)
serviceability limit state
strength limit of a component corresponding to a defined limit of permanent (or inelastic) deformation of this component which leads to damage to the system if it is exceeded

Note 1 to entry: This limit is also called the serviceability limit state in building codes based on limit states design.

3.1.5
damage state (of the system)
state where the system needs repairing because one of its components has exceeded its damage limit

Note 1 to entry: The system needs repairing because it is not capable of fulfilling its task under design loads or because design clearances may be reduced (e.g. conductor to ground).

3.1.6
elements
different parts of a component

Note 1 to entry: For example, the elements of a steel lattice tower are steel angles, plates and bolts.

3.1.7
exclusion limit
***e* %**
value of a variable taken from its distribution function and corresponding to a probability of *e* % of not being exceeded

3.1.8
failure limit (of a component)
ultimate limit state
strength limit of a component which leads to the failure of the system if this limit is exceeded

Note 1 to entry: If this strength limit is exceeded, the system will reach a state called “ultimate limit state” as defined in building codes based on limit states design.

3.1.9
failure state (of the system)
state of a system in which a major component has failed because one of its components has reached its failure limit (such as by rupture, buckling, overturning)

Note 1 to entry: This state leads to the termination of the ability of the line to transmit power and needs to be repaired.

3.1.10
intact state
state in which a system can accomplish its required function and can sustain limit loads

3.1.11
limit load
Q_T
climatic load corresponding to a return period, T, used for design purposes without additional load factors

Note 1 to entry: Refer to 5.2.1.

3.1.12
load factor
γ
factor to be multiplied by the limit load in order to design line components

3.1.13
operating period
general measure of useful (or economical) life

Note 1 to entry: Typical operating periods of transmission lines vary from 30 years to 80 years.

3.1.14
reference wind speed
V_R
wind speed at 10 m in height, corresponding to an averaging period of 10 min and having a return period T

Note 1 to entry: When this wind speed is taken in a terrain type B, which is the most common case in the industry, the reference wind speed is identified as V_{RB}.

3.1.15
reference ice load
g_R **or** t_R
reference limit ice loads (g_R is a unit ice weight and t_R is a uniform radial ice thickness around the conductor) having a return period T

3.1.16
reliability (structural)
probability that a system performs a given task, under a set of operating conditions, during a specified time

Note 1 to entry: Reliability is thus a measure of the success of a system in accomplishing its task. The complement to reliability is the probability of failure or unreliability.

3.1.17
return period (of a climatic event)
T
average occurrence in years of a climatic event having a defined intensity

Note 1 to entry: The inverse of the return period is the yearly frequency which corresponds to the probability of exceeding this climatic event in a given year.

3.1.18
safety
ability of a system not to cause human injuries or loss of lives

Note 1 to entry: In this document, safety relates mainly to protection of workers during construction and maintenance operations. The safety of the public and of the environment in general is covered by national regulations.

3.1.19
security (structural)
ability of a system to be protected from a major collapse (cascading effect) if a failure is triggered in a given component

Note 1 to entry: Security is a deterministic concept as opposed to reliability which is a probabilistic concept.

3.1.20
strength factor
Φ
factor applied to the characteristic strength of a component

Note 1 to entry: This factor takes into account the coordination of strength, the number of components subjected to maximum load, quality and statistical parameters of components.

3.1.21
system
set of components connected together to form the transmission line

3.1.22
task
function of the system (transmission line), i.e. to transmit power between its two ends

3.1.23
unavailability
inability of a system to accomplish its task

Note 1 to entry: Unavailability of transmission lines results from structural unreliability as well as from failure due to other events such as landslides, impact of objects, sabotage, defects in material, etc.

3.1.24
use factor
U
ratio of the actual load (as built) to limit load of a component

Note 1 to entry: For tangent supports, it is virtually equal to the ratio of actual to maximum design spans (wind or weight) and for angle supports; it also includes the ratio of the sines of the half angles of deviation (actual to design angles).

3.2 Symbols and abbreviations

a	Unit action of wind speed on line elements (Pa or N/m^2)
A_c	Wind force on conductors (N)
A_i	Wind force on insulators (N)
A_t	Wind force acting on a tower panel made of steel angles, A_tc for cylindrical tower members (N)
B_i	Reduction factor of the reference wind speed for wind and ice combinations
C_x	Drag coefficient (general form)
C_i	Drag coefficient of ice covered conductors (C_iL for low probability and C_iH for a high probability)
C_xc	Drag coefficient of conductors
C_xi	Drag coefficient of insulators
C_xt	Drag coefficient of supports C_xt1, C_xt2 for each tower face (C_xtc on cylindrical tower members)
COV	Coefficient of variation, also identified as v_x (ratio of standard deviation to mean value)

d	Conductor diameter (m)
d_{tc}	Diameter of cylindrical tower members (m)
D	Equivalent diameter of ice covered conductors (D_H for high probability and D_L for low probability) (m)
e	Exclusion limit (%)
e_N	Exclusion limit of N components in series (%)
$F_{(x)}$	Cumulative distribution function of variable x
G	Wind factor (general form)
G_c	Combined wind factor of conductors
G_t	Combined wind factor of towers
G_L	Span factor for wind calculations
g	Unit weight of ice (N/m)
g_m	Yearly maximum ice load (N/m)
$\overline{g}_m$	Mean yearly maximum ice loads (N/m)
g_{max}	Maximum weight of ice per unit length observed during a certain number of years (N/m)
g_R	Reference design ice weight (N/m)
g_H	Ice load having a high probability (N/m)
g_L	Ice load having a low probability (N/m)
H	Horizontal tensile load
K_R	Terrain roughness factor
K_d	Diameter factor related to the influence of conductor diameter
K_h	Height factor to be multiplied by $\overline{g}$ to account for the influence of height above ground
K_n	Factor to be multiplied by $\overline{g}$ to account for the influence of the number of years with icing observations
l_e	Length of a support member (m)
L	Span length or wind span (m)
L_m	Average span (m)
n	Number of years of observation of a climatic event
N	Number of components subjected to maximum loading intensity
Q	General expression used to identify the effects of weather related loads on lines and their components
Q_T	The system limit load corresponding a return period T
q_0	Dynamic reference wind pressure due to reference wind speed V_R (q_{0L}, q_{0H} for low and high probability) (Pa or N/m^2)
Re	Reynolds number
R	Strength (usually in Pa or in kN depending on components)
$\overline{R}$	Mean strength (units same as for R)
R_c	Characteristic strength (units same as for R)
$(e)R$	Exclusion limit (e) of strength (units same as for R)
RSL	Residual static load of a broken conductor (kN)
S_i	Projected area of insulators (m^2)

S_t Projected area of a tower panel (m^2)
t Ice load expressed in uniform radial ice thickness around the conductor (mm)
t_R Reference ice load expressed in uniform radial thickness around the conductor (mm)
T Return period of weather events (years)
u Number of standard deviations between mean strength and characteristic strength
U Use factor
v_x Coefficient of variation (COV) of variable x
V Wind speed (m/s)
V_m Yearly maximum wind speed (m/s)
$\bar{V}_m$ Mean yearly maximum wind speed (m/s)
V_G Yearly maximum gradient wind speed (m/s)
$\bar{V}_G$ Mean yearly maximum gradient wind speed (m/s)
V_R Reference wind speed (m/s)
V_{iL} Low probability wind speed associated with icing (m/s)
V_{iH} High probability wind speed associated with icing (m/s)
w Unit weight of conductor or ground wire (N/m)
$\bar{x}$ Mean value of variable x
Y Horizontal distance between foundations of a support (m)
z Height above ground of conductors, centre of gravity of towers panels, or insulator strings (m)
γ Load factor (general form)
γ_U Use factor coefficient
γ_{TW} Load factor to adjust the 50 year wind speed to a return period T
γ_{Tit} Load factor to adjust the 50 year ice thickness to a return period T
γ_{Tiw} Load factor to adjust the 50 year ice weight to a return period T
δ Ice density (kg/m^3)
Φ Strength factor (general form)
Φ_R Global strength factor
Φ_N Strength factor due to number of components subjected to maximum load intensity
Φ_S Strength factor due to coordination of strength
Φ_Q Strength factor due to quality
Φ_c Strength factor related to the characteristic strength R_c
σ_x Standard deviation of variable x
σ_g Standard deviation of yearly maximum ice loads (N/m)
μ Mass of air per unit volume (kg/m^3)
τ Air density correction factor
ν Kinetic air viscosity (m^2/s)

Ω Angle between wind direction and the conductor (degrees)

θ Angle of incidence of wind direction with the tower panel (degrees)

θ' Angle of incidence of wind direction with cylindrical elements of tower (degrees)

χ Solidity ratio of a tower panel

4 General

4.1 Objective

This document serves either of the following purposes:

a) It provides design criteria for overhead lines based on reliability concepts. The reliability based method is particularly useful in areas where significant amounts of meteorological and strength data are readily available. This method may however be used for lines designed to withstand specific climatic loads, either derived from experience or through calibration with existing lines that had a long history of satisfactory performance. In these cases, design consistency between strengths of line components will be achieved, but actual reliability levels may not be known, particularly if there has been no evidence or experience with previous line failures.

   It is important to note that the design criteria in this standard do not constitute a complete design manual for transmission lines. However, guidance is given on how to increase the line reliability if required, and to adjust the strength of individual components to achieve a desired coordination of strength between them.

b) It provides a framework for the preparation of national standards for transmission lines using reliability concepts and employing probabilistic or semi-probabilistic methods. These national standards will need to establish the climatic data for the use and application of this standard in addition to other data specific to each country.

   This standard also provides minimum safety requirements to protect people and construction/maintenance personnel from injury, as well as to ensure an acceptable level of service continuity (safe and economical design).

The design criteria in this standard apply to new line conditions. It is however a fact of life that transmission lines age and lose strength with time. The amount of strength reduction due to ageing is difficult to generalize, as it varies from one component to another, and also depends on the type of material, the manufacturing processes and the environmental influences. This issue is currently being studied by relevant technical bodies.

The requirements are specified in this standard, while, in Annexes A to G, additional informative data and explanations are given.

4.2 System design

The methodology is based on the concept whereby a transmission line is designed as a system made of components such as supports, foundations, conductors and insulator strings. This approach enables the designer to coordinate the strengths of components within the system and recognizes the fact that a transmission line is a series of components where the failure of any component could lead to the loss of power transmitting capability. It is expected that this approach should lead to an overall economical design without undesirable mismatch.

As a consequence of such a system design approach, it is recognized that line reliability is controlled by that of the least reliable component.

An overhead transmission line can be divided into four major components as shown in Figure 1. Subsequently, each component may be divided into different elements.

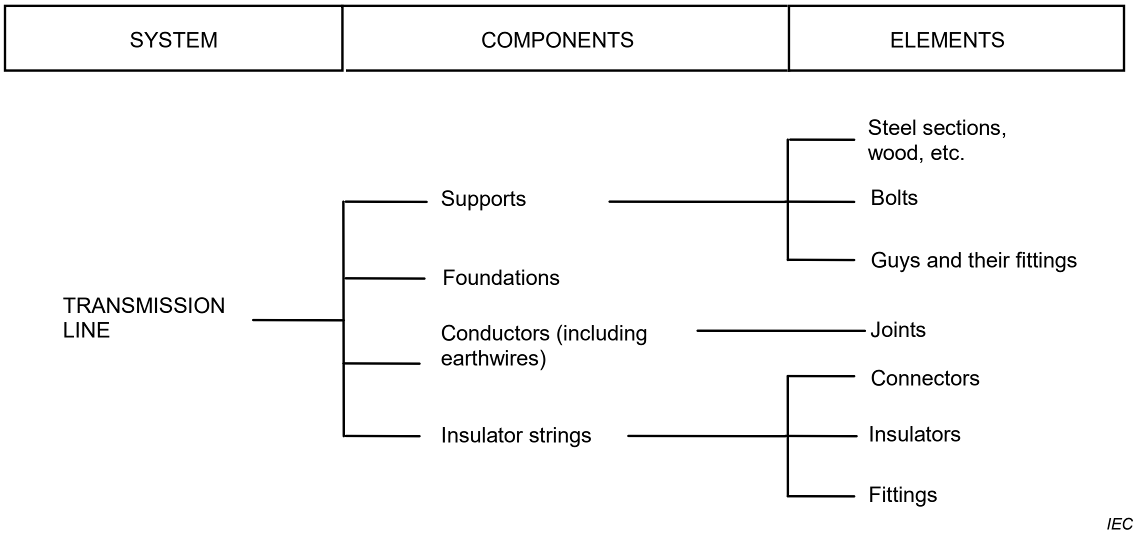

Figure 1 – Diagram of a transmission line

4.3 System reliability

The objective of design criteria described in this standard is to provide for reliable and safe lines. The reliability of lines is achieved by providing strength requirements of line components larger than the quantifiable effects of specified weather related loads. These climatic loads are identified in this standard as well as means to calculate their effects on transmission lines. However, it has to be recognized that other conditions, not usually dealt with in the design process, can occur and can lead to line failure such as impact of objects, defects in material, etc. Some measures, entitled security requirements, included in this standard provide lines with enough strength to reduce damage and its propagation, should it occur.

5 General design criteria

5.1 Methodology

5.1.1 General

The recommended methodology for designing transmission line components is summarized in Figure 2 and can be described as follows:

a) Collect preliminary line design data and available climatic data.

NOTE 1 In some countries, reference wind speed, such as the 50 year return period, is given in national standards.

b1) Select the reliability level in terms of return period of limit loads.

b2) Select the security (failure containment) requirements.

b3) Identify safety requirements imposed by mandatory regulations and construction and maintenance loads.

c) Calculate climatic variables corresponding to selected return period of limit loads.

d1) Calculate climatic limit loads on components.

d2) Calculate loads corresponding to security requirements.

d3) Calculate loads related to safety requirements during construction and maintenance.

e) Determine the suitable strength coordination between line components.

f) Select appropriate load and strength factors applicable to load and strength formulas.

g1) Calculate the characteristic strengths required for components.

g2) Check compliance with national safety regulations, and adjust the design if non-compliant.

h) Design line components for the above strength requirements.

This standard deals with items b) to g2). Items a) and h) are not part of the scope of this standard. They are identified by a shaded frame in Figure 2.

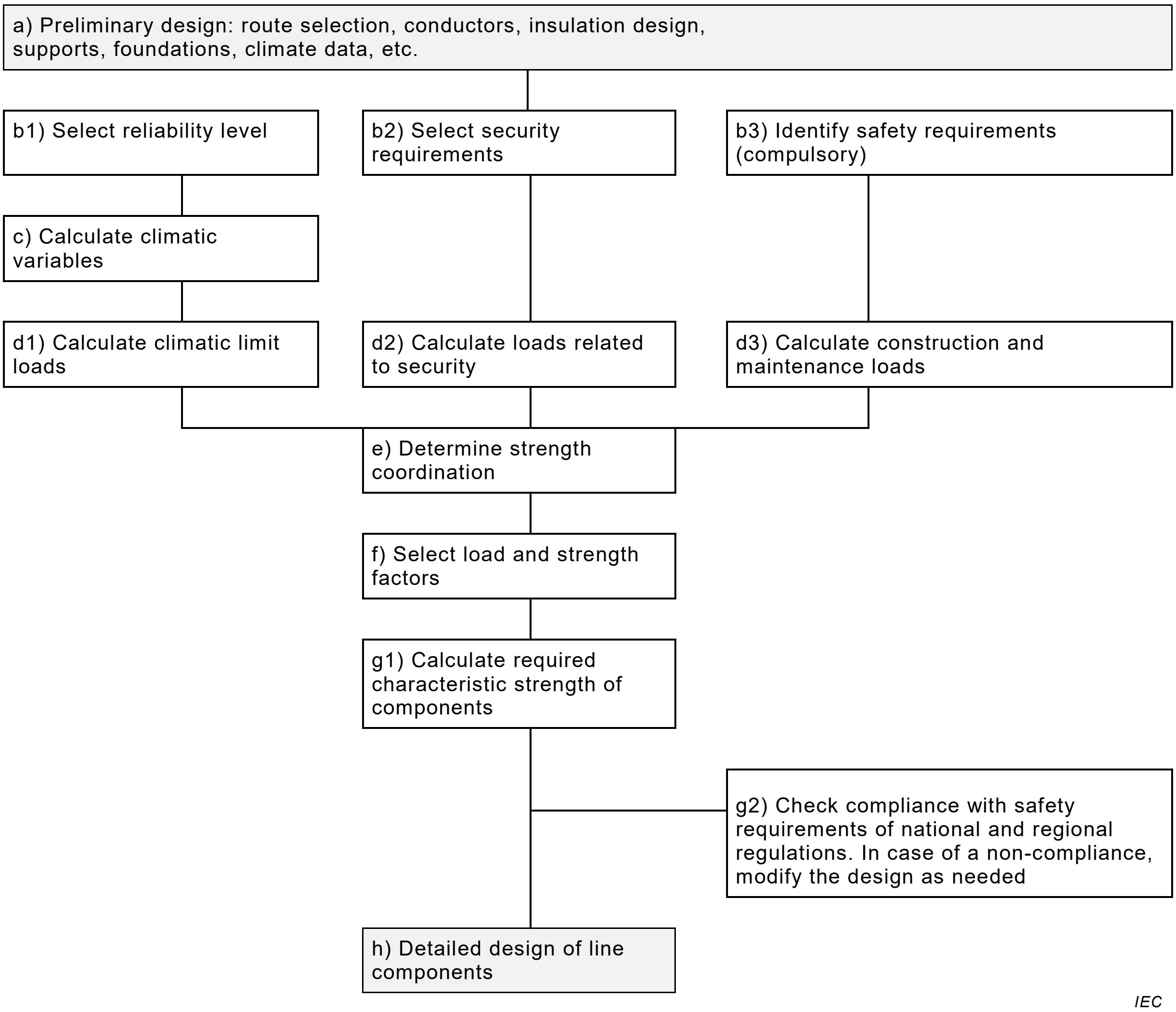

Figure 2 – Transmission line design methodology

5.1.2 Reliability requirements

5.1.2.1 Reliability levels (weather related loads)

Reliability requirements aim to ensure that lines can withstand the defined climatic events (wind, ice, ice and wind, having a return period T) and the loads derived from these events during the projected life cycle of the system and can provide service continuity under these conditions.

Transmission lines can be designed for different reliability levels (or classes). For the purposes of this standard, the reference reliability level is defined as the reliability of a line designed for a 50 year return period climatic event associated with a 10 % exclusion limit of strength (applies to the components selected as the least reliable). This reference reliability level is generally regarded as providing an acceptable reliability level in respect of continuity of service and safety.

Lines can be designed for higher reliability levels by increasing the return period T of climatic events. A higher reliability can be justified for example by the importance of the line in the network. Three reliability levels are proposed in this standard and are assumed to cover the range of values to be considered for most transmission lines. These levels are expressed in terms of return periods of climatic events as shown in Table 1. For temporary lines, some wooden poles or lines of limited importance, return periods of about 25 years may be appropriate.

Table 1 – Reliability levels for transmission lines

Reliability levels	1	2	3
T, return period of climatic event, in years	50	150	500

NOTE Some national regulations and/or codes of practice, sometimes impose, directly or indirectly, design requirements that may restrict the choices offered to designers.

Other values of T in the range of 50 to 500 years, such as 100, 200 and 400 years, can be used if justified by local conditions.

In some cases, individual utility's requirements can dictate other reliability levels depending on the proper optimization between initial cost of the line and future cost of damage, as well as on uncertainties related to input design parameters.

5.1.2.2 Approximate values for yearly reliability

Both loads (Q) and strengths (R) are stochastic variables and the combined reliability is computable if the statistical functions of load Q and strength R are known. The condition for a line to be reliable is when loads effects are less than the strength withstand of the line. The reliability and probability of failure are complements to 1 as indicated by the following Formula (1).

$$\text{Yearly reliability (probability of survival)} = 1 - \text{yearly probability of failure} \qquad (1)$$

When the characteristic strength, deemed to be the strength being exceeded with 90 % probability (i.e. the exclusion limit is 10 %, or (10 %)R), is set equal to the climatic load corresponding to the selected return period T, designated Q_T , various probabilistic combinations lead to a theoretical yearly minimum reliability of around (1 – 1/2T). This association between load and strength is given by the following Formula (2)[1]:

$$Q_T = (10\ \%)\ R \qquad (2)$$

Throughout the present standard, the loading Q_T is called the system limit load having a return period T.

The actual reliability can be different if load and strength data are not sufficiently accurate or available. In the latter case, the absolute reliability may not be known, but its value relative to a reference design may be computed if new line parameters are comparable to the reference values.

The relationship expressed in Formula (2) can be further refined through the introduction of correction factors related to the following items:

1 Additional information and background data related to reliability based design of overhead lines can be found in the following CIGRÉ publications and brochures: Technical Brochures 109 and 178 (Chapter 2), *Electra* papers 1991-137 and 2000-189.

- use factors of components: the fact that all components are not used at their maximum design parameter (wind span, weight span, height of support, line angle) contributes to an increase of the reliability;
- characteristic strength R_C: in actual lines, the characteristic strength of most components corresponds to an exclusion limit less than 10 %. If, in such cases, it is assumed to be equal to 10 %, then the actual reliability of the line will be higher;
- strength coordination: a selected strength coordination results in an increase of strength or withstand resistance of some components;
- number of components subjected to maximum loading intensity: whenever a storm or severe icing occurs, not all structures will be subjected to maximum loads, since the storm is limited in spatial extension;
- quality control during fabrication and construction: by these measures, low quality material will be eliminated. No components with strengths below a certain limit will be used;
- critical wind direction: in case of wind loads it generally is assumed for design purposes that maximum wind velocities also act in the most unfavourable direction. However, maximum winds are distributed in angle sectors. Approximate calculations carried out by the CIGRE Working Group SC 22.06 showed that more realistic assumptions could reduce the probability of failure by one order of magnitude.

In practice, if the above factors are not properly taken into account in Formula (2), then the resulting reliability will be different from the theoretical values.

While the above-mentioned factors contribute to the actual reliability usually being higher than the theoretical values, other factors could lead to opposite effects, i.e. a reduction in reliability. For example, the ageing of some line components and the fatigue due to a large number of loading cycles will have a negative effect on reliability.

It is noted that requirements for other events such as earthquakes are not covered in this standard.

NOTE Usually, transmission structures are not affected by earthquake loads because they are already designed for important horizontal wind and longitudinal loads that are applied on the higher points of the structure.

It is also noted that the above probability of failure is only one of the components of the total line unavailability as described in 3.1.23.

5.1.3 Security requirements

Security requirements correspond to special loads and/or measures intended to reduce probability of uncontrollable progressive (or cascading) failures that may extend well beyond an initial failure. These measures are detailed in 6.6.

NOTE Some security measures, such as those providing longitudinal strength of broken conductor loads for failure containment can also lead to an increase in reliability to withstand unbalanced ice loads.

5.1.4 Safety requirements

Safety requirements consist of special loads for which line components (mostly support members) have to be designed, to ensure that construction and maintenance operations do not pose additional safety hazards to people. These measures are detailed in 6.5.

5.2 Load-strength requirements

5.2.1 Climatic loads

Loads associated with climatic events are random variables. Three weather-related loading conditions are recognized: wind, ice, and wind and ice combined. When statistical data of wind and/or ice are available, these can be used to compute the limit load Q_T, for each component exposed to the climatic event under consideration.

In the calculation process for each component, the following condition has to be checked:

$$\text{Design limit load} < \text{design strength} \quad (3)$$

or, more precisely,

Load factor $\gamma \times$ effect of limit load Q_T < strength factor Φ $\times$ characteristic strength R_c.

With the approach proposed, system limit loads Q_T are used for design without additional load factors. Consequently, γ is taken equal to 1.

Thus the previous relation becomes:

$$\text{effect of } Q_T < \Phi \times R_c \quad (4)$$

For weather related loads, the effects of Q_T are detailed in 6.2 to 6.4.

Formula (5) is used to compute the minimum value of characteristic strength R_c for each component in order to withstand limit loads.

$$R_c > (\text{effect of } Q_T) / \Phi \quad (5)$$

Limit load Q_T can be obtained from the statistical analysis of climatic data in accordance with techniques detailed in Annexes C and D. In some national standards, a reference (usually a 50 year return period value) climatic variable is specified. In such a case, the climatic variable for any return period T (years) can be estimated by multiplying the 50 year reference value of the climatic variable by the load factor γ_T, given in Table 2. In case other types of distribution functions are found to better represent the variation of yearly extreme values, the γ_T factors below may change.

Table 2 – Default γ_T factors for adjustment of climatic loads in relation to return period T versus 50 years

	Wind speed	Ice variable		
Return period T years	γ_{TW}	γ_{Tit} **(ice thickness)**	**or**	γ_{Tiw} **(ice weight)**
50	1	1		1
150	1,10	1,15		1,20
500	1,20	1,30		1,45

NOTE The above γ values are sufficiently accurate for a COV of up to 16 % for wind speed, 30 % for ice thickness and 65 % for unit ice weight and are derived from the Gumbel distribution function.

5.2.2 Design requirements for the system

Three types of design conditions shall be checked: reliability, security and safety. Table 3 summarizes the context of loads, the required performance and the strengths limit states associated with each condition.

Table 3 – Design requirements for the system

Condition (or requirement)	Type of load	Required performance	Corresponding limit state
Reliability	Climatic loads due to wind, ice, ice plus wind, and temperature, having a return period T	To ensure reliable and safe power transmission capability	Damage limit
Security	Torsional, vertical, and longitudinal loads	To reduce the probability of uncontrollable propagation of failures (failure containment)	Failure limit
Safety	Construction and maintenance loads	To ensure safe construction and maintenance conditions	Damage limit

5.2.3 Design formula for each component

When designing individual line components, Formula (4) can be expanded into:

$$\gamma_{\mathrm{U}} \times \text{effect of } Q_{\mathrm{T}} < \Phi_{\mathrm{R}} \times R_{\mathrm{c}} \tag{6}$$

γ_{U} is the use factor coefficient. It is derived from the distribution function of the use factor U and expresses the relationship between effective (actual) and design (original) conditions or parameters. The use factor U is a random variable equal to the ratio of the effective (actual line conditions) limit load applied to a component by a climatic event to the design limit load for this component under the same climatic event (using maximum parameters). Symbol γ_{U} is introduced because components are designed in general by families, not individually for each support and location. Thus, since components are usually designed prior to specific knowledge of their real line parameters (wind and weight spans and angle of deviation), it is admissible to use γ_{U} = 1 for new lines design.

NOTE This is equivalent to considering that design is controlled by the maximum span in the line for a given support type.

It is important to note this simplification will certainly have a positive influence on reliability. However, the influence of γ_{U} on reliability can be fully considered for existing lines, where use parameters of components are fully known. For a detailed discussion on the subject, refer to Annex E.

R_{c} is the characteristic strength. It is the value guaranteed in appropriate standards for new components, usually with a 90 % to 98 % probability. This value is also called the guaranteed strength, the minimum strength or the minimum failing load. When not specified or calculated, the exclusion limit of R_{c} can be conservatively taken as 10 % (typical values are in the range of 2 % to 10 %). It is generally accepted that line components will age with time, just like any structural components, and will suffer a reduction in their strength. This quantification of loss of strength with time is not covered in this standard and the reliability values suggested herein are based on new line conditions. If the reduction of strength due to ageing or fatigue can be quantified in some components, it is needed to define the minimum residual strength that should trigger the replacement of these components.

Very often, standards only provide a single normative value usually associated with failure of the component, while the design approach mentioned above requires the consideration of two limits: damage and failure limits. If the damage limit corresponding to R_{c} is not specified in the standards, Tables 18 to 21 can be used to provide such values.

Φ_R is a global strength factor applicable to the component being designed that takes into account:

a) features related to the system
   - the number (N) of components exposed to the limit load Q_T during any single occurrence of this load event, (hence Φ_N);
   - the coordination of strengths selected between components, (hence Φ_S).

b) features related to the component
   - the difference in the quality of the component during prototype testing and actual installation, (hence Φ_Q);
   - the difference between the actual exclusion limit of R_c and the supposed e = 10 %, (hence Φ_c).

As these factors are assumed statistically independent:

$$\Phi_R = \Phi_N \times \Phi_S \times \Phi_Q \times \Phi_c \tag{7}$$

The above strength factors Φ are detailed in 7.2.

6 Loadings

6.1 Description

This clause defines structural loadings considered for the design of transmission line components.

As indicated in 5.2.1, three load categories are considered:

a) loads due to climatic events or any loads derived from them which govern the reliability of the line for the expected life time.

   These loads will be analysed in the following subclauses:
   - wind loads (6.2);
   - ice without wind (6.3);
   - ice with wind (6.4);

b) loads related to safety requirements (construction and maintenance) (6.5);

c) loads related to security requirements (failure containment) (6.6).

6.2 Climatic loads, wind and associated temperatures

6.2.1 General

This subclause defines the procedures to evaluate the wind and associated temperature effects on line components and elements (conductors, insulator strings, supports).

6.2.2 Field of application

Although this subclause applies in principle to any overhead line, it is most accurately defined for the following conditions:

- Span lengths between 200 m and 800 m. Calculations of the various coefficients (in particular for gusty winds) have to be checked for span lengths outside this range. However, for span lengths greater than 800 m, a gust coefficient corresponding to 800 m span could be safely chosen. For span lengths less than 200 m, the values applicable to 200 m span can be applied.

- Height of supports less than 60 m. Taller supports could be designed following the same principles, but the calculated wind actions would need to be checked. In particular, the eigen frequency of structures above 60 m will often increase the gust response factor.
- Altitude of crossed areas not exceeding 1 300 m above the average level of the topographic environment, except where specific study results are available.
- Terrain without local topographical features whose size and shape are likely to significantly affect the wind profile of the region under consideration.

It is important to note that requirements for special winds associated with localized events such as tornadoes are not part of the normative requirements in this standard. It is however recognized that these winds can cause serious damage to transmission lines either directly (due to wind forces) or indirectly (due to impact of debris carried by wind)[2]. Furthermore, the effects of acceleration due to funnelling between hills are not covered and may require specific climatic studies to assess such influences. In the informative Annex G, proposals are made to deal with wind acceleration due to local topography such as slopes.

6.2.3 Terrain roughness

Wind speed and turbulence depend on the terrain roughness. With increasing terrain roughness, turbulence increases and wind speed decreases near ground level. Four types of terrain categories, with increasing roughness values, are considered in this standard as indicated in Table 4.

Table 4 – Classification of terrain categories

Terrain category	Roughness characteristics
A	Large stretch of water upwind, flat coastal areas
B	Open country with very few obstacles, for example airports or cultivated fields with few trees or buildings
C	Terrain with numerous small obstacles of low height (hedges, trees and buildings)
D	Suburban areas or terrain with many tall trees

It is noted that snow accumulation on the ground will reduce the ground roughness and may increase the terrain category to a higher one in Table 4. However, this should also take into account the duration of such snow and the occurrence of very high wind speeds during snow presence

In areas where trees could be cut, care should be taken in the selection of the terrain category.

6.2.4 Reference wind speed V_R

The reference wind speed V_R (m/s) corresponding to the selected return period T is defined as mean value of the wind during a 10 min period at a level of 10 m above ground.

Usually V_R is measured in weather stations typical of terrain type B. In such cases, V_R is identified as V_{RB}.

2 CIGRE WG B2.06 has published Technical Brochure No. 350 on special localized high intensity wind phenomena such as tornadoes and downbursts. The proposals of this Brochure will benefit from validation by service experience and are referred to in order to encourage additional research and comprehension about the impact of HIW on overhead transmission lines. These studies suggest some simplified support load cases that could reduce the probability of failure of structures subjected to such wind events. It is however important to note that many of these high wind events involve flying debris that can lead to support failures irrespective of the proposed strengthening.

If the reference wind speed for terrain category B, V_{RB} is only known, V_R can be determined with

$$V_R = K_R \; V_{RB} \tag{8}$$

where K_R is the roughness factor in Table 5.

When available wind data differs from these assumptions, refer to 6.2.5 for conversion methods.

6.2.5 Assessment of meteorological measurements

Wind action is evaluated on the basis of the reference wind speed V_R that can be determined from a statistical analysis of relevant wind speed data at 10 m above ground and with an averaging period of 10 min.

Usually, meteorological stations (except those along the coast or in urban areas) are placed in areas of B terrain category, such as airports.

Nevertheless, the meteorological wind speed may be recorded in a terrain category × site at 10 m above the ground as a mean value over a period of time t in s. Let $V_{x,t}$ be this speed. If it is not measured at 10 m height above ground, the data should be adjusted first to this reference height of 10 m.

The variation of V in terms of height was not taken into account, as anemometers are, most of the time, placed at a height of about 10 m above surrounding ground. If this height z (m) differs from 10 m, the variation of wind speed with height z can be derived from the so-called "power law", shown in Formula (9). The value of α is found in Table 5.

Table 5 – Factors describing wind action depending on terrain category

Factor	Terrain category			
	A	B	C	D
α	0,10 to 0,12	0,16	0,22	0,28
K_R	1,08	1,00	0,85	0,67

$$V_z = V_R \left(\frac{z}{10}\right)^{\alpha} \tag{9}$$

Or more generally:

$$V_{z1} = V_{z2} \left(\frac{z1}{z2}\right)^{\alpha} \tag{10}$$

The curves of Figure 3 enable to determine the ratio $V_{x,t}$ / $V_{x,10\ min}$ as a function of the averaging period for each category of roughness at the location of the meteorological site. These values may be used in the absence of local data or studies.

NOTE Some countries have recently switched to a 2 s or 3 s averaging period. The conversion from the 2 s or 3 s wind speeds to the 10 min wind can be derived from available wind statistics, or if lacking, from Figure 3.

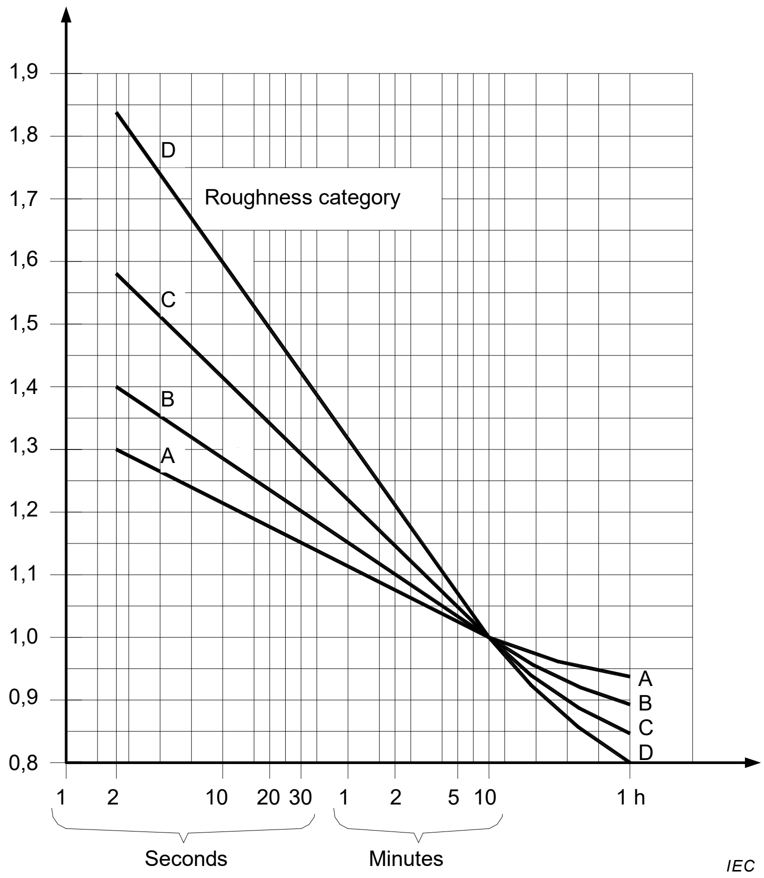

Figure 3 – Relationship between meteorological wind velocities at a height of 10 m depending on terrain category and on averaging period

6.2.6 Determination from gradient wind velocities

Where meteorological stations are remote from the locations considered for the erection of the line, the gradient wind speed, defined as the speed at the level on the top of the earth's boundary layer, which is 800 m to 1 000 m above ground, may be used as a basis for assessment of design wind velocities.

The gradient wind action is characterized by the mean value of yearly maximum gradient wind velocities $\overline{V}_G$ and its standard deviation σ_{VG}. From the wind speed $\overline{V}_G$ the mean of the yearly maxima $\overline{V}_m$ (10 m above ground) can be approximated by the following formula:

$$\overline{V}_m \text{ (B)} = 0{,}5\ \overline{V}_G \qquad (11)$$

Data for $\overline{V}_G$ can usually be obtained from national weather services.

6.2.7 Combination of wind speed and temperatures

Unless a strong positive correlation is established between wind speed and temperature, it is assumed that reference wind speed V_R does not usually occur with minimum temperature. Consequently, only two combinations of wind speed and temperature shall normally be considered for design purposes; the first being reference wind speed combined with average daily minimum temperature and the second being reduced wind speed combined with extreme minimum temperature.

In practice, the following two combinations need to be checked:

a) High wind speed at average temperature condition

The wind velocity V_R defined above shall be considered as occurring at an air temperature equal to the average of the daily minimum temperatures, peculiar to the site. If statistical data confirms that high winds occur at a different temperature, then the statistical value shall be used.

b) Reduced wind speed at the minimum low temperature condition

1) Reduced wind speed

The reduced wind speed is equal to the reference wind speed V_R multiplied by a coefficient chosen according to local meteorological conditions. When there is no reliable knowledge of local conditions, a value of 0,6 for this coefficient is suggested.

2) Temperature associated with the reduced wind speed

The minimum temperature shall be considered as being equal to the yearly minimum value, having a return period of T years.

It is noted that the design of transmission lines is not generally controlled by the combination of reduced wind speed and minimum low temperatures (condition b2 above). This loading case may therefore be omitted, except for cases of supports with short spans (typically less than 200 m) and minimum low temperatures (typically below –30 °C), or in the case of supports with dead-end insulators.

6.2.8 Number of supports subjected in wind action, effect of length of line

Gusts with maximum wind speed are limited in width. An individual gust will therefore hit only one support and the adjacent spans. Nevertheless, to take care of the several gusts with approximately the same magnitude, it is proposed to assume that five supports are hit in flat or rolling terrain and two in mountains.

For long lines, the probability to be hit by extreme wind actions is higher than for short lines. The effect depends on many aspects, such as variation of terrain and climate, design of supports adjusted to the terrain and the loads to be expected there. The design of the line should aim at the same reliability of the total line related to the service life of the line.

Lines with relatively short length up to 100 km could be designed for a reliability level as proposed in 5.1.2.1. For longer lines, in order not to increase the probability of failure, the return periods of chosen design assumptions should be extended so as to achieve the overall reliability. The adjustment of return periods is not required if the map of wind data has already been adjusted to take into account the space covered by service area.

6.2.9 Unit action of the wind speed on any line component or element

The characteristic value a of the unit action in Pa (N/m²), due to the wind blowing at right angles to any line component or element (conductors, insulator strings, all or part of the support) is given by the following formula:

$$a = q_0\, C_x\, G \tag{12}$$

where q_0 is the dynamic reference wind pressure (in Pa or N/m²) and is given in terms of the reference wind speed V_{RB} modified by roughness factor K_R (see Table 5) corresponding to the terrain category at the location of the line:

$$q_0 = \frac{1}{2}\tau\mu\left(K_R V_{RB}\right)^2 \quad (V_{RB} \text{ in m/s, and } q_0 \text{ in N/m}^2) \tag{13}$$

where

μ is the air mass per unit volume equal to 1,225 kg/m^3 at a temperature of 15 °C and an atmospheric pressure of 101,3 kPa at sea level;

τ is the air density correction factor. When limit wind speeds are known to be strongly correlated with an altitude and/or temperature significantly different from the assumptions of 15 °C and sea level, the correction factor τ given in Table 6 can be applied to the pressure q_0, otherwise, τ is considered to be equal to 1;

C_x is the drag (or pressure) coefficient depending on the shape and surface properties of the element being considered;

G is the combined wind factor, taking into account the influences of the height of the element above ground level, terrain category, wind gusts and dynamic response (component effect). In the case of conductor loads, this factor shall be split into two factors G_L and G_c. while in the case of supports and insulators this factor is identified as G_t.

These factors shall be considered separately for each line component or element.

Table 6 – Correction factor τ of dynamic reference wind pressure q_0 due to altitude and temperatures

Temperature °C	**Altitude** m			
	0	**1 000**	**2 000**	**3 000**
30	0,95	0,84	0,75	0,66
15	1,00	0,89	0,79	0,69
0	1,04	0,94	0,83	0,73
–15	1,12	0,99	0,88	0,77
–30	1,19	1,05	0,93	0,82

NOTE The reference value corresponds to 0 m altitude and a temperature of 15 °C. Interpolation between the above factors is acceptable.

6.2.10 Evaluation of wind loads on line components and elements

6.2.10.1 Wind loads on conductors

Wind effects on conductors consist of loads due to wind pressure as well as the effect of the increase in the mechanical tension.

The load (A_c) in N due to the effect of the wind pressure upon a wind span L, applied at the support and blowing at an angle Ω with the conductors, is given by the following expression, using q_0 of Formula (13).

$$A_c = q_0\, C_{xc}\, G_c\, G_L\, d\, L \sin^2 \Omega \quad (14)$$

where

C_{xc} is the drag coefficient of the conductor taken equal to 1,00 for the generally considered stranded conductors and wind velocities. Other values can be used if derived from direct measurements or wind tunnel tests. It is noted that more evidences support a C_x of 1,2 for EW or conductors having a diameter of 15 mm or less.

G_c is the combined wind factor for the conductors given in Figure 4, which depends on height z and terrain categories.

G_L is the span factor given in Figure 5.

d is the diameter of the conductor (m).

L is the wind span of the support, equal to half the sum of the length of adjacent spans of the support.

Ω is the angle between the wind direction and the conductor (Figure 7).

The total effect of the wind upon bundle conductors shall be taken as equal to the sum of the actions on the sub-conductors without accounting for a possible masking effect of one of the sub-conductors on another.

The height to be considered for conductors is the center of gravity of the suspended conductor theoretically located at the lower third of the sag. For the purpose of transmission support calculations, it is acceptable to consider z equal to the height of attachment point of the conductor at the support (for horizontal configuration) or of the middle conductor (for vertical circuit configuration). These assumptions for conductors are conservative and compensate for the increased height of the ground wire on top of the support.

Values different from Figures 4 and 5 can be used if supported by data and validated models.

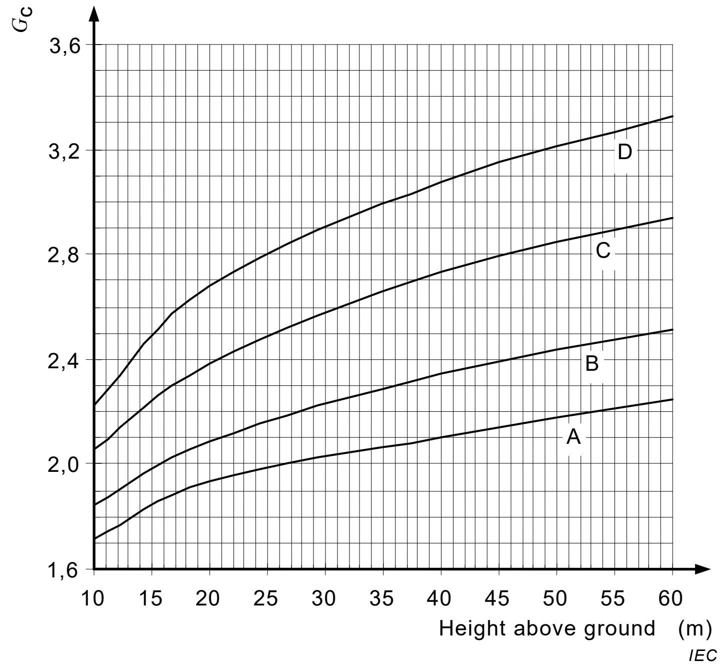

Figure 4 – Combined wind factor G_c for conductors for various terrain categories and heights above ground

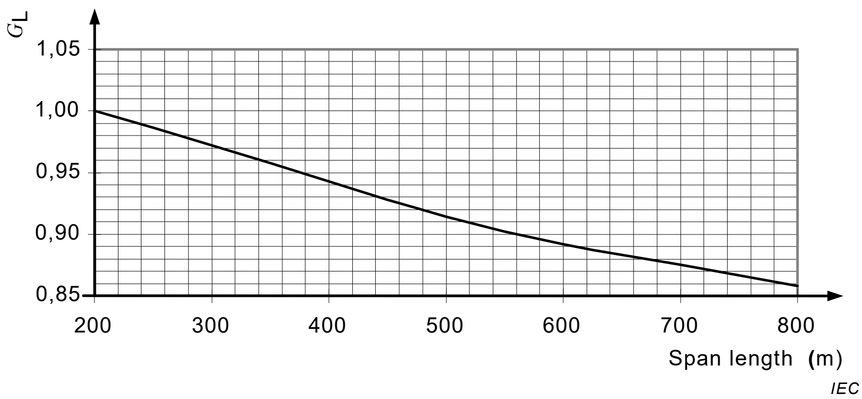

Figure 5 – Span factor G_L

NOTE Formulas for Figures 4 and 5 are given in Annex B.

6.2.10.2 Wind effect on conductor tension

Wind acting on conductors will cause an increase in their mechanical tension that can be computed with standard sag-tension methods. Two cases of wind and temperature combinations shall be checked, as stated in 6.2.7.

If a series of spans is separated by suspension insulators, the ruling span concept may be used for tension calculations. It is important to note that the ruling span concept implies that the same wind pressure applies to all spans between dead-end insulators. This assumption becomes more conservative with an increasing number of suspension spans and length of insulator strings. In such cases, the conductor tension due to wind load calculated with Formula (14) can be reduced, if supported by experience or data, but in no case by more than 20 %. With regard to ground wires, no reduction of wind pressure is applicable because the absence of suspension insulator strings prevents equilibrium of horizontal tensions even at suspension supports, hence, results in the inapplicability of the ruling span concept.

The ruling span of a series of suspension spans between dead-ends is equal to $(\Sigma L^3/\Sigma L)^{1/2}$.

Caution should be exerted when using the above reduction factor of up to 20 % because some supports may be used in sections with few suspension spans and even as a single span between dead-end supports; in such case, no reduction factor is applied.

It is important to properly control any damaging vibration to conductors by limiting the conductors tensions to appropriate levels (refer to Annex F for further information).

6.2.10.3 Wind loads on insulators strings

Wind loads acting on insulator strings originate from the load A_c transferred by the conductors and from the wind pressure acting directly on the insulator strings. The latter load is applied conventionally at the attachment point to the support in the direction of the wind and its value (in N) is given by:

$$A_i = q_0\, C_{xi}\, G_t\, S_i \tag{15}$$

where

q_0 is the dynamic reference wind pressure in Pa (N/m^2);

C_{xi} is the drag coefficient of the insulators, considered equal to 1,20;

G_t is the combined wind factor given in Figure 6, variable with the roughness of the terrain, and with the height of the centre of gravity of the insulator string above the surrounding land. The same average height of conductors can be used.

NOTE The formula for Figure 6 is given in Annex B.

S_i is the area of the insulator string projected horizontally on a vertical plane parallel to the axis of the string (m^2). In the case of multiple strings, the total area can be conservatively taken as the sum of all strings.

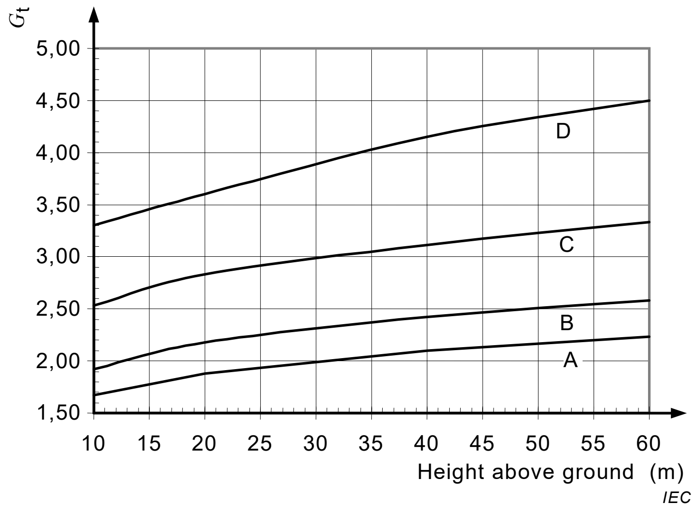

Figure 6 – Combined wind factor G_t applicable to supports and insulator strings

It is noted that wind on insulator strings has a small effect on design of supports. Consequently, it may be acceptable for most lines to simplify the calculation of wind pressure by conservatively adopting the same pressure as the one applied to supports.

6.2.10.4 Wind loads on supports

6.2.10.4.1 General

Wind loads on the supports consist of the wind loads transmitted by conductors and insulators as well as the wind loads acting on the support itself.

The method of determination of wind loadings on the support itself is only given in this standard for the most common types of supports, i.e. lattice towers and towers with cylindrical elements. This method can, however, be applied to other types of supports.

During detailed design of supports, an iterative process is required in order to compute wind loads on supports. This is due to the fact that the projected area of members is only known after completion of the support detailed design.

6.2.10.4.2 Lattice towers of rectangular cross-section

Two methods are proposed for calculating loads on lattice towers. The first method is based on a ‘panel’ concept where the same pressure is applied to the windward face of the panel based on its calculated solidity ratio, and the second method is based on wind pressure being applied individually to all tower members taking into account the angle of incidence of wind with the normal to the tower.

<u>Method 1: wind on panels</u>

In order to determine the effect of the wind on the lattice tower itself, the latter is divided into different panels. Panel heights are normally taken between the intersections of the legs and bracing and typically having a height of 10 m to 15 m.

For a lattice tower of square/rectangular cross-section, the wind loading A_t (in N), in the direction of the wind, applied at the centre of gravity of this panel, made up of various support members, is equal to:

$$A_t = q_0\,(1 + 0{,}2\sin^2 2\theta)\,(S_{t1}\,C_{xt1}\cos^2\theta + S_{t2}\,C_{xt2}\sin^2\theta)\,G_t \qquad (16)$$

where

q_0	is the dynamic reference wind pressure Pa (N/m^2), see Formula (13);
θ	is the angle of incidence of the wind direction with the perpendicular to face 1 of the panel in a horizontal plane (Figure 7);
S_{t1}	is the total surface area projected normally on face 1 of the panel (m^2);
S_{t2}	is the total surface area projected normally on face 2 of the panel (m^2);
C_{xt1} , C_{xt2}	are the drag coefficients peculiar to faces 1 and 2 for a wind perpendicular to each face. C_{xt1} , C_{xt2} are given in Figure 8 for panels of the tower where all or some of the members exposed have plane surfaces, and in Figure 9 where all support members have a circular section;
χ	is the solidity ratio of a panel equal to the projected area of members divided by the total panel area. The solidity ratio χ of one face is the ratio between the total surface of the support members (S_{t1} or S_{t2}), defined above, and the circumscribed area of the face of the considered panel;
G_t	is the combined wind factor for the supports given in Figure 6. The height above ground is measured at the centre of gravity of the panel.

NOTE The projections of the bracing elements of the adjacent faces and of the diaphragm bracing members can be neglected when determining the projected surface area of a face.

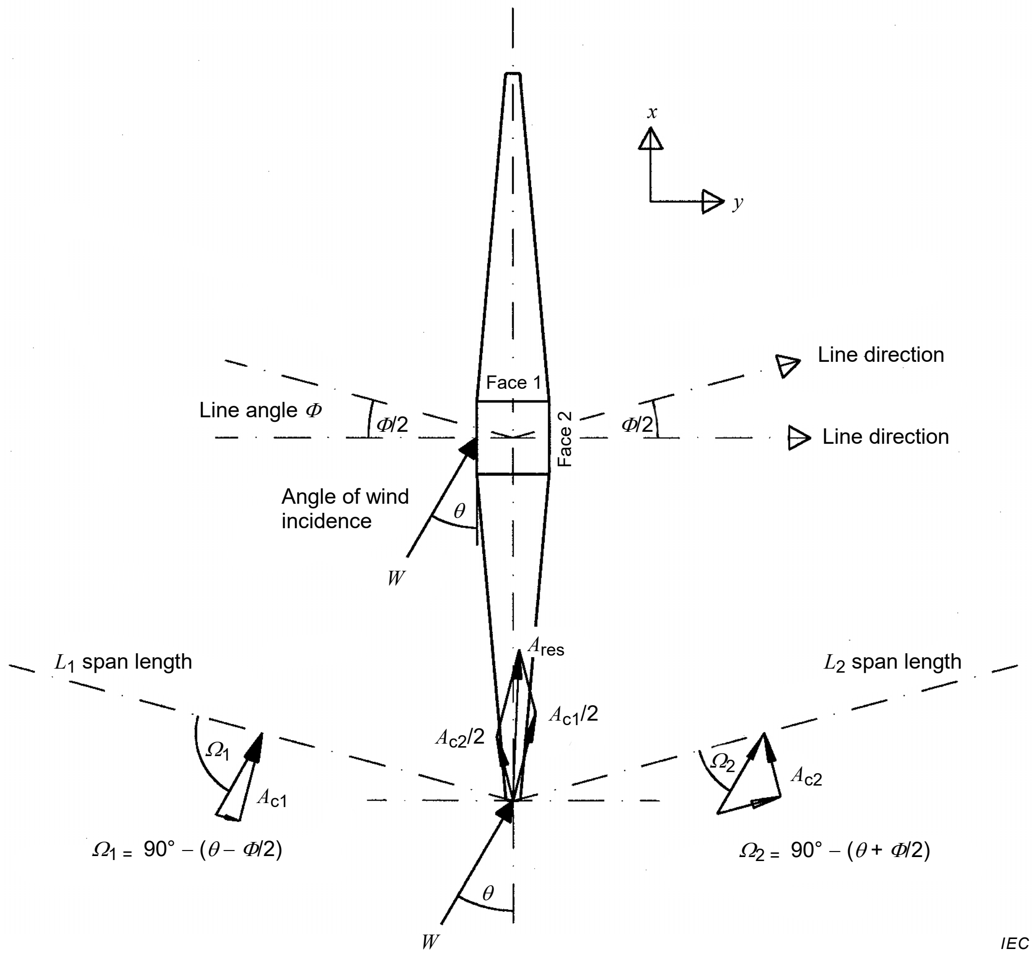

Figure 7 – Definition of the angle of incidence of wind

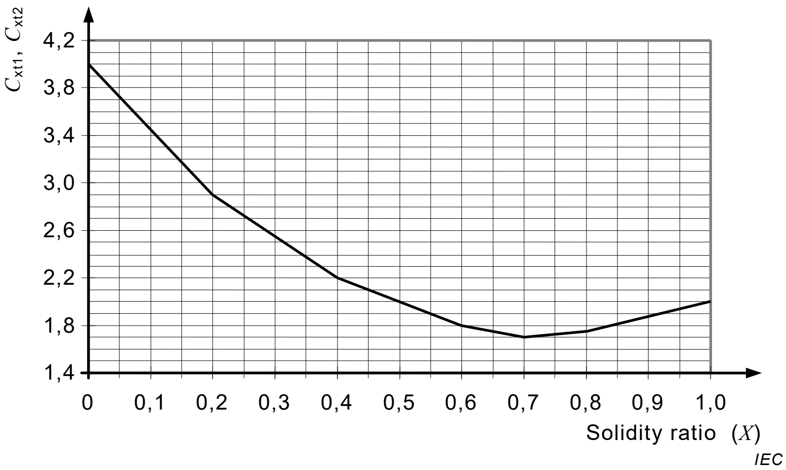

Figure 8 – Drag coefficient C_{xt} for lattice supports made of flat sided members

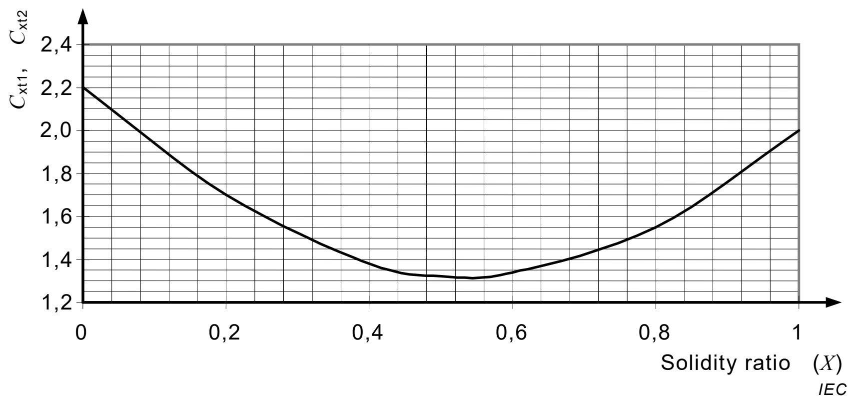

Figure 9 – Drag coefficient C_{xt} for lattice supports made of rounded members

NOTE 1 The formulas for Figures 8 and 9 are given in Annex B.

Method 2: Wind on all tower members

In this method, the wind force on each member is calculated independently (neglecting shielding) based on the geometrical relationship between the wind velocity vector and the axis of the member. The force is in the plane formed by the wind velocity vector and the member axis, and it is perpendicular to the member. The wind force in N is calculated using the formula below.

$$A_t = q_0 \, C_{xt} \, G_t \, S_t \sin^2 \Omega \qquad (17)$$

where

q_0 is the dynamic reference wind pressure Pa (N/m^2), see Formula (13);

S_t is the surface of the flat member exposed to wind (m^2);

C_{xt} is the drag coefficients peculiar to a flat surface if the tower members are made of steel angles. In such a case, C_{xt} can be considered equal to 1,6. In the case of tower members made of round tubes, C_{xt} can be considered equal to 1,0. In both cases, the shielding is neglected and this load is calculated on all tower members;

NOTE 2 This method will provide conservative results compared to Method 1, but is a logical way of taking into account the complexity of a more accurate calculation of wind effects of supports.

G_t is the combined wind factor for the supports given in Figure 6. The height above ground is measured at the centre of gravity of the member;

Ω is the angle between the wind direction and the tower steel member. This angle is calculated from the 3-d geometry, between the direction of the wind velocity vector and the member axis in the plane formed by the wind velocity vector and the member axis.

This method is particularly suitable for software implementation.

6.2.10.4.3 Supports with cylindrical members having a large diameter (d_{tc} > 0,2 m)

For such supports the effect of the wind loading (in N) in the direction of the wind, on each member l_e long, applied at the centre of gravity of the member, is equal to:

$$A_{tc} = q_0\, C_{xt}\, G_t\, d_{tc}\, l_e \sin^3 \theta' \tag{18}$$

where

θ' is the angle formed by the direction of the wind and the cylinder axis;

d_{tc} is the diameter of the cylinder (m);

l_e is the length of the member (m);

G_t is the combined wind factor, a function of the terrain category and the height h of the centre of gravity of the member above the ground (Figure 6);

C_{xtc} is the drag coefficient for a wind perpendicular to the axis of the cylinder. The value of C_{xtc} depends on the Reynolds number Re corresponding to the gust speed at this height, and on the roughness of the cylinder. An acceptable simplification is to consider the most unfavourable case of a rough cylinder. The value of C_{xtc} is given in Figure 10 in terms of Re that corresponds to the reference wind speed V_R at this height z (corrections with height are described in Formula (8)) and is given by:

$$\mathrm{Re} = \frac{d_{tc} \times V_R}{\nu} \tag{19}$$

ν is the kinetic air viscosity (ν = 1,45 × 10^{-5} m^2/s at 15 °C)

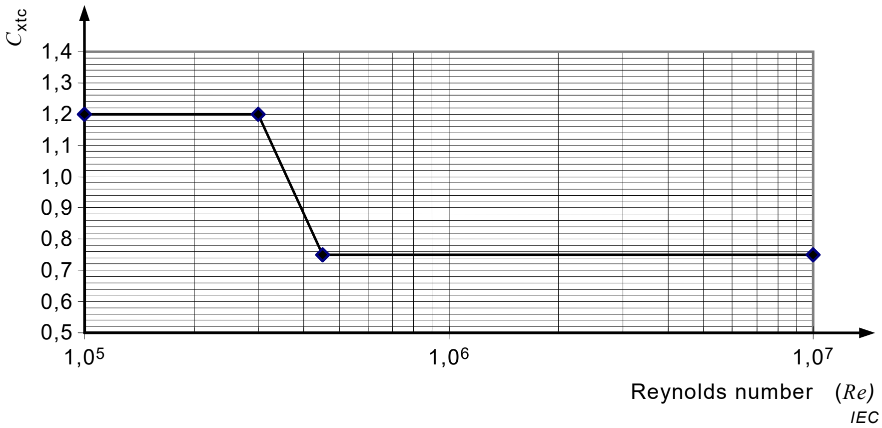

Figure 10 – Drag coefficient C_{xtc} of cylindrical elements having a large diameter

In the case of wood pole, it is acceptable to simplify the wind load calculation on the poles by adopting a C_x value equal to 1. This simplification is due to the fact that the height of wood poles is generally limited and does not warrant a precise calculation of wind on the pole itself.

In the case of structures (steel poles and frames) made of polygonal cross section, the values of C_x can be found in Table 7:

Table 7 – Drag coefficient of polygonal pole sections

Member shape	Drag coefficient, C_x
16-sided polygonal	0,9
12-sided polygonal	1,0
8-sided polygonal	1,4
6-sided polygonal	1,4
Square, rectangle	2,0

6.2.10.4.4 Lattice towers of triangular cross-section

Formula (16) can be used, except that the drag coefficient C_{xt} shall be calculated from Table 8 as a function of the solidity ratio χ.

Table 8 – Drag coefficient of structures having a triangular section

Solidity ratio, χ	Drag coefficient C_{xt} for triangular-section structures
<0,025	3,6
0,025 to 0,44	3,7 to 4,5χ
0,45 to 0,69	1,7
0,70 to 1,00	1,0+χ

NOTE Above Table 7 and Table 8 are taken from ASCE 74, 2006 draft.

6.3 Climatic loads, ice without wind

6.3.1 Description

Ice loads consist of all combinations of frozen water that adheres to transmission line components such as freezing rain, in-cloud-icing, wet snow, etc. (see description in Annex C). This standard covers two main types of icing: precipitation icing and in-cloud icing.

In mountains or regions where both types of icing may occur, the different data for the two types may be treated separately, with separate distributions to provide the basis for the design load. If a difference between the design loads for the two types of ice is apparent, the less important may be ignored, and the more important may take care of combined occurrences.

Although significant loadings due to the presence of ice also involve some wind during and after an icing event, ice only is first considered here to establish reference conditions that will serve as a basis for the wind and ice combined loadings given in 6.4 as well as non-uniform ice conditions described in 6.3.6.4.

6.3.2 Ice data

Ice load is a random variable that is usually expressed either as a weight per unit length of conductor g (N/m), or as a uniform radial thickness t (mm) around conductors and ground wires. In real conditions, ice accretion is random in both shape and density and depends on the type of accretion as indicated in Figure C.1. However, for ease of calculations, these are converted to an equivalent radial ice thickness (t) around conductors with a relative density δ of 0,9. Formula (20) expresses the relation between g and t:

$$g = 9{,}82 \times 10^{-3}\ \delta\ \pi\ t\ (d + t/1\ 000) \qquad (20)$$

where

g is the ice weight per unit length (N/m);

δ is the ice density (kg/m^3);

t is the radial ice thickness, assumed uniform around the conductor (mm);

d is the conductor diameter (m).

For an ice density δ = 900 kg/m^3, Formula (20) becomes:

$$g = 27{,}7\ t\ (d + t/1\ 000)$$

When both t and d are expressed in mm and δ = 900 kg/m^3, Formula (20) becomes:

$$g = 0{,}027\ 7\ t\ (t + d) \tag{21}$$

with g in N/m.

Ice load should ideally be deduced from measurements taken from conductors and locations representative of the line. These measurement techniques are described in IEC 61774. Ice accretion models can also supplement direct ice data measurement, but require appropriate validation with real data.

A very important factor with ice accretion is the effect of the terrain. It is usually rather difficult to transfer knowledge acquired from one site to another because the terrain strongly influences the icing mechanism.

For design purposes, icing data from measuring stations near or identical to the line site are ideally required. Very often, this will not be the case and service experience with existing installations will provide additional input.

Ice accretion on structures should be considered (refer to C.9.2 for a suggested method).

NOTE It is noted that weight of ice on lattice steel structures can be quite significant and can reach or exceed the weight of the structure itself in case of radial ice thickness greater than 30 mm to 40 mm. Furthermore, icing on towers also increase the exposed area to wind, hence, loads due to wind on ice covered towers.

6.3.3 Evaluation of yearly maximum ice load by means of meteorological data analysis

Sufficient data for using the statistical approach in this standard may be obtained by means of an analysis of available standard weather or climatic data over a period of 20 years or more, combined with at least five years of ice observation on the transmission line sites. For cases where years of icing data is less than 20, an approximate method is given in 6.3.4.

If a reliable ice accretion model is available to estimate values for yearly maximum ice loads during a certain number of years, this model can be used to generate ice data which will be used in the statistical analysis. Information about the line site which is necessary to validate and adjust the predicting model may be taken from past experience with existing transmission or distribution lines, from field observations or from the effect of icing on vegetation.

Such a predicting model can be rather simple or become sophisticated, depending on icing severity, terrain, local weather, number or types of ice data collecting sites.

6.3.4 Reference limit ice load

6.3.4.1 Based on statistical data

The reference design ice load g_R, or t_R if ice thickness is chosen as the ice variable, are the reference limit ice loads corresponding to the selected return period T (function of the reliability level of the line). The g_R or t_R values can be directly obtained from the statistical analysis of data obtained either from direct measurements, icing models, or appropriate combinations of both.

NOTE 1 The figures and formulas given in this subclause are based on g_R (N/m) being the ice variable. However, Formula (20) can be used to convert from g_R to t_R if the latter is chosen as the ice variable.

If records of yearly maximum ice loads g_m during a period of at least 10 years are available, the mean value $\overline{g}_m$ is derived from the records of the yearly maximum ice load g_m; the standard deviation σ_g is calculated or estimated according to Table 9.

Table 9 – Statistical parameters of ice loads

Number of years with observation n	Mean value $\overline{g}_m$	Standard deviation σ_g
Only maximum icing value g_{max} of yearly maximum ice loads g_m is known	$\overline{g}_m = 0{,}45\ g_{max}$	$\sigma_g = 0{,}5\ \overline{g}_m$
$10 \leq n \leq 20$	$\overline{g}_m$	$0{,}5\ \overline{g} \leq \sigma_g \leq 0{,}7\ \overline{g}_m$

If data is measured (or model simulated) on conductor diameters and heights typical of the line, there will not be any further adjustment to this value. However, if data is measured at the assumed reference height of 10 m on a 30 mm conductor diameter, g_R should be adjusted by multiplying it with a diameter factor K_d and a height factor K_h applicable to the actual line conditions.

K_d is given in Figure 11.

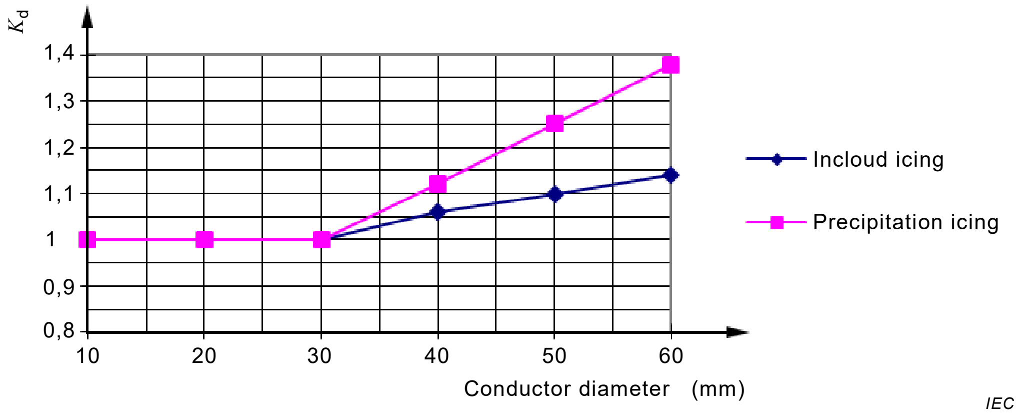

Figure 11 – Factor K_d related to the conductor diameter

For both types of icing, when $K_d\ \overline{g}$ exceeds 100 N/m, the value of K_d is no longer increased. If $\overline{g}$ (average of yearly maximum values of g) is above 100 N/m and d greater than 30 mm, K_d is considered equal 1,0.

K_h describes the variation of g with the height of conductors above the ground. Its value is given in Figure 12.

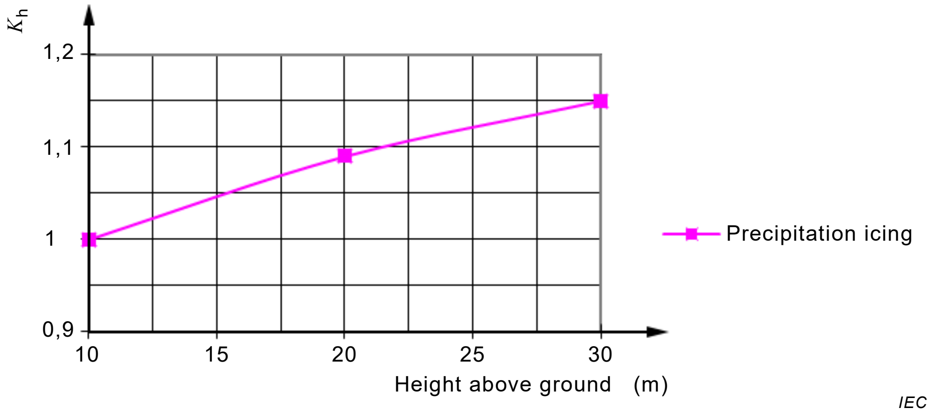

Figure 12 – Factor K_h related to the conductor height

As a simplification, it is suggested that the value g_R be the same for phase conductors and ground wires in the same span, but there is growing evidence that the higher ground wire may accumulate more ice for some types of ice accretion. For variation of in-cloud icing accretion with height, refer to the note in C.9.1.

NOTE 2 Some recent studies suggest that bundled conductors may collect less wet snow or in-cloud ice than single conductors due to the difference in torsional behaviour. This matter is currently under investigation.

NOTE 3 As regards wet snow, the thickness of icing may be considered the same on conductors and earth-wires unless service experience indicates otherwise.

6.3.4.2 Based on service experience

Where icing data or reliable ice accretion models are not available, the only alternative is to rely on service experience based on actual ice loads observed on the conductors or deduced from failure events. In both cases, neither the return period of the ice loads, nor the level of reliability will be known.

6.3.5 Temperature during icing

The default temperature to be considered with ice conditions shall be –5 °C, except for cases of incloud icing where the temperature can be in the range of –20 to –5 °C or where icing records confirm that lower temperatures occur during icing persistence on conductors.

6.3.6 Loads on support

6.3.6.1 General

Three different icing conditions on the conductors shall be considered when determining the loads on the support without wind. These are considered to be the most significant for torsional and flexural loads of supports and encompass the majority of the icing conditions that are likely to occur:

- uniform ice formation: weight condition;
- non-uniform ice formation: longitudinal and transverse bending condition;
- non-uniform ice formation: torsion condition.

Note that ice loading conditions combined with wind are considered in 6.4 and are considered important for transverse loads on supports.

6.3.6.2 Loading cases description

In the description of the different loading conditions, the values of the ice loads are given as functions of the reference design ice load g_R. It is important to be aware of the fact that g_R may vary from one span to another in a section of a line, due to local terrain effects, giving non-uniform situations. The aim is to propose conventional loading conditions for the purpose of calculating support loads which are typical for known occurrences of ice loading.

When computing loads on a support from conductors, the effects of the swing of the insulator set, deflection or rotation of the support and/or foundations and the interaction with other conductors shall be considered. Sometimes, simplifying assumptions or load cases can be used if these result in conservative load cases.

Ice may not accumulate or shed uniformly from adjacent spans. A non-uniform ice formation is defined as an ice load corresponding to the probability of an ice accretion on typically three spans or more on one side of the support, whilst on the other spans in the clause the ice is reduced to a certain percentage of that value.

NOTE Unbalanced ice loads due to unequal accretion or ice shedding will invariably occur during icing events. Statistics of unbalanced ice loads are not usually available; however, the recommendations given in this standard should be sufficient to simulate typical unbalanced ice loads that occur in such conditions.

6.3.6.3 Uniform ice formation – Maximum weight condition

The maximum uniform ice loading on the conductors is assumed to occur, when the conductor ice loading is equivalent to the reference limit ice load (g_R). The overload per unit length is g_R (N/m), and the total conductor load per unit length = $w + g_R$ (w is the unit weight of conductors in N/m).

6.3.6.4 Non-uniform ice conditions on phase conductors and ground wires

Unequal ice accumulations or shedding in adjacent spans will induce critical out-of-balance longitudinal loads on the supports. Unbalanced ice loads can occur either during ice accretion, e.g. in-cloud icing with significant changes in elevation or exposure, or during ice shedding.

Suggested configurations of non-uniform icing conditions are described in Table 10 for support types shown in Figure 13.

a) Single circuit support **b) Double circuit support**

Figure 13 – Typical support types

NOTE For multi-circuit lines, the number of phases subject to non-uniform ice can be different, but not less than that given for double circuit lines.

Table 10 – Non-uniform ice loading conditions

Type of supports	Longitudinal bending condition		Transverse bending condition		Torsional condition	
	Left span	Right span	Left span	Right span	Left span	Right span
Single circuit	xyabc	XYABC	xYabC	xYabC	XYabC	XYABC
Double circuit	xabcdef	XABCDEF	XabcDEF	XabcDEF	XabcDEF	XABCDEF

NOTE In this table, the letters A, B, C, D, E, F, X, Y represent conductors and spans loaded with 0,7 g_R, while the letters a, b, c, d, e, f, x, y represent conductors and spans loaded with 0,4 × 0,7 g_R. Factors 0,7 and 0,4 are suggested and other values can be used as substantiated by experience.

Where the exposure of the line to its surroundings changes from one span to another, unbalanced loads larger than those described above should be considered. During calculations of longitudinal loads on structures due to unbalanced ice loads, the flexibility of structures and insulator movement shall be taken into account to calculate the resulting longitudinal forces. Use of simplified assumptions is accepted as long as they lead to conservative results.

Where specific sections of an OHL are exposed to severe in-cloud icing and adjacent spans have different levels of moisture-laden winds, it may be applicable to consider maximum ice loading on one side of the support and bare conductors on the other side.

6.4 Climatic loads, combined wind and ice loadings

6.4.1 General

The combined wind and ice loadings treated in this subclause relate to wind on ice-covered conductors. Wind on ice-covered supports and insulator strings can, if necessary, be treated in a similar way with special attention to drag coefficients.

6.4.2 Combined probabilities – Principle proposed

The action of wind on ice-covered conductors involves at least three variables: wind speed that occurs with icing, ice weight and ice shape (effect of drag coefficient). This action results in both transverse and vertical loads.

Ideally, statistics of wind speed during ice presence on conductors should be used to generate the combined loadings of ice and wind corresponding to the selected reliability level. Since detailed data and observations on ice weight, ice shape and coincident wind are not commonly available, it is proposed to combine these variables in such a way that the resulting load combinations will have the same probability of occurrence or return periods T as those adopted for each reliability level.

Assuming that maximum loads are most likely to be related to combinations involving at least one maximum value of a variable (either of wind speed, ice weight or ice shape), a simplified method is proposed: a low probability-high value (index L) of a variable is combined with high probability-low values (index H) of the other two variables, as is shown in Table 11. This simplification is equivalent to associating one variable (e.g. ice load) having a return period T with the average of yearly values of all the other variables related to this loading case, such as wind during icing or the drag coefficient.

Table 11 – Return period of combined ice and wind load

Reliability level	Return period T years	Return period of the variable having a low probability of occurrence (index L)	Return period of remaining variables (index H)
1	50	50	Average of yearly maximum values
2	150	150	Average of yearly maximum values
3	500	500	Average of yearly maximum values

The density of ice varies with the type of icing and it is recommended that low density ice be combined with the high probability drag coefficient and vice-versa.

Usually, the combination of a low probability drag coefficient (highest value of C_{iH} , or C_{iL}) with a high probability ice and a high probability wind does not constitute a critical loading case. However, if previous service experience or calculation confirms that this load combination can be critical, it should be considered for design purposes.

Consequently, two loading combinations will be considered in this standard: Low ice probability (return period T) associated with the average of yearly maximum winds during icing presence, and low probability wind during icing (return period T) associated with the average of the yearly maximum icing. Details about these two loading cases are given in 6.4.7.2.

The low probability (reference values) of ice or wind has already been dealt with separately in the previous paragraphs. These should correspond to the return period T selected for design purposes.

With regard to wind, it is important to note that wind data to be considered is when icing is present on conductors. Such data may not usually be available and it is generally accepted to deduce it from the yearly wind statistics.

6.4.3 Determination of ice load

The two main types, precipitation and in-cloud icing, require a separate determination of the maximum ice load associated with wind.

If there is almost no data on combined wind and ice, it can be assumed that $g_L = g_R$ and g_H = 0,40 g_R. If combined wind and ice data are available, statistical methods can be used to estimate values for combined variables corresponding to the selected return period T or to the average of yearly maximum winds.

6.4.4 Determination of coincident temperature

The default temperature to be considered for combined wind and ice conditions shall be –5 °C for all types of icing, except for in-cloud icing where the temperature can be in the range of –20 to –5 °C. In both above cases, the default value can be replaced by data, if available.

6.4.5 Determination of wind speed associated with icing conditions

6.4.5.1 Freezing rain (precipitation icing)

Wind velocities associated with icing episodes can be calculated from data, if available or, when there is little or no data, from the following assumptions. In the latter case, the reference wind speed is multiplied by a reduction factor B_i as indicated below:

$$V_{iL} = B_i \times V_R \tag{22}$$

where B_i = (0,60 to 0,85).. This range of B_i is assumed to correspond to the reference wind speed (T = 50, 150 or 500 years) during icing persistence on conductors.

$$V_{iH} = B_i \times V_R \tag{23}$$

where B_i = (0,4 to 0,5). This range of B_i is assumed to correspond to the average of yearly maximum wind speed during icing persistence on the conductors.

The given range of values in the above formulas represents typical values of wind speed during icing periods and takes into account the relative rarity of maximum wind speed during icing periods.

When combined data are available, the process described for wind or ice loading can be used to select a value corresponding to a return period T for each of the expected types of icing.

When wind speed data is not strictly correlated with icing, one should determine the associated maximum wind speed by using the yearly maximum wind speed recorded during freezing precipitation and the following period whilst the air temperature remains below 0 °C (suggested maximum period 72 h).

6.4.5.2 Wet snow (precipitation icing)

Based on both local meteorological conditions and experience, the reduction in the wind speed (V_R) can be determined in a similar manner to that described for freezing rain (see 6.4.5.1). In the absence of specific experience or data, it is suggested to use the same reduction factors as for freezing rain.

6.4.5.3 Dry snow (precipitation icing)

In the absence of any specific data for dry snow, the same values stated for wet snow may be used.

6.4.5.4 Hard rime (in-cloud icing)

In certain areas, hill tops, for example, the maximum rime ice accretion on the conductors usually occurs with the maximum wind speed associated with in-cloud icing. However, in other areas the maximum ice accretion usually occurs under relative low wind speeds.

Basic meteorological and terrain information should be used to evaluate the probability of severe in-cloud icing along the line route, and the corresponding data should be introduced in the calculations. Otherwise, the values given for freezing rain may be used.

6.4.6 Drag coefficients of ice-covered conductors

Wherever possible, drag coefficients for ice covered conductors should be based on actual measured values. In the absence of this data, the effective drag coefficients and ice densities are given in Table 12. In the absence of reliable field observations and data, the upper values of ice densities in Table 12 shall be used.

Table 12 – Drag coefficients of ice-covered conductors

	(Precipitation) Wet snow	(In-cloud) Soft rime	(In-cloud) Hard rime	(Precipitation) Glaze ice
Effective drag coefficient C_{iH}	1,0	1,2	1,1	1,0
Associated ice density δ (kg/m^3)	300 to 600	300 to 600	600 to 900	900

The effective drag coefficient is a multiplying factor on the assumed cylindrical shape for the specified ice volume (see Table 14). There is evidence to support the increase in the drag coefficient for ice covered conductors for two reasons: the first due to the effect of the equivalent diameter and the second due to the ice shape itself as opposed to the round and smooth cylinder.

NOTE The uniform thickness of ice around the conductor corresponds to the minimum overall diameter, i.e. the most compact projected area.

It is assumed that the value of C_i is the same for the ice coverings related to T = 50, 150, and 500 years.

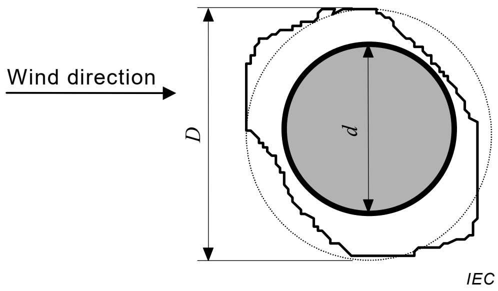

Figure 14 – Equivalent cylindrical shape of ice deposit

6.4.7 Determination of loads on supports

6.4.7.1 Unit action of the wind on the ice-covered conductors

With reference to 6.2.6, the characteristic value (a) of the unit wind action on ice covered conductors with the wind blowing horizontally and perpendicular to the line is given by the formula:

$$a = q_0\ C_i\ G_c\ G_L \tag{24}$$

$$q_{0L} = \frac{1}{2}\mu\tau K_R^2 V_{iL}^2 \quad \text{or} \quad q_{0H} = \frac{1}{2}\mu\tau K_R^2 V_{iH}^2 \quad \text{Pa (N/m}^2\text{)}$$

dependent on the loading condition, and with appropriate $C_i = C_{iL}$ (see Table C.4 in Annex C);

G_c is the combined wind factor of conductors as defined in 6.2.10.1;

G_L is the span factor as defined in 6.2.10.1;

τ is the density correction factor given in 6.2.9.

6.4.7.2 Loads on supports

Two combined wind and ice loading conditions should be considered with their coincident vertical loading.

The load (A_c) in N due to the effect of the wind upon a wind span L, applied at the support and blowing at an angle Ω with the conductors, is given by the following expression, using q_0 of Formula (13).

NOTE 1 The wind span L of a support is equal to half the sum of the length of adjacent spans.

$$A_c = q_0\ C_i\ G_c\ G_L\ D\ L\ \sin^2\Omega$$

For the two recommended loading conditions, the wind force on ice-covered conductors shall be:

- Condition C1 (highest value of ice load to be combined with average of yearly maximum wind speed during ice persistence):

$$A_{c1}=q_{0H}\ C_{iH}\ G_c\ G_L\ D_L\ L\ \sin^2\Omega \tag{25}$$

With $D_L = (d^2 + 4g_L/9{,}82\pi\delta)^{0{,}5}$

- Condition C2 (highest value of wind speed during ice persistence to be combined with average of yearly maximum ice load):

$$A_{c2} = q_{0L}\ C_{iH}\ G_c\ G_L\ D_H\ L\ \sin^2\Omega \tag{26}$$

With $D_H = (d^2 + 4g_H/9{,}82\pi\delta)^{0{,}5}$

NOTE 2 In Annex C, Table C.4, there is a third combination called C3 that is mentioned but the above two conditions were found, in general, to be most critical. Should the 3rd condition be required, the information contained in Annex C may be used.

In the above formulas, D_L, D_H are diameters (m) of the equivalent cylindrical shapes for the types of ice being considered.

where

g_L and g_H	is the ice load (N/m) corresponding respectively to low and high probabilities of occurrence;
δ	is the highest density for type of ice being considered (kg/m^3);
Ω	is the angle between wind direction and the conductor.

Where support members are critical for lower conductor vertical loads at the supports, the effect of reduced vertical loads and the presence of aerodynamic lift forces should be considered. It is suggested that the lift force per unit length is not likely to exceed 50 % of the drag force per unit length of ice covered conductors.

6.5 Loads for construction and maintenance (safety loads)

6.5.1 General

Construction and maintenance operations are the occasions when failure of a line component is most likely to cause injury or loss of life. These operations should be regulated to eliminate unnecessary and temporary loads which would otherwise demand expensive reinforcing of all supports, especially in ice-free areas.

National regulations and/or codes of practice generally provide minimum safety rules and requirements with respect to public safety.

In addition, construction and maintenance loading cases will be established in this standard as recommended hereafter. The system stress under these loadings shall not exceed the damage limit, and the strength of the supports shall be verified either by testing (see IEC 60652) or by reliable calculation methods.

6.5.2 Erection of supports

The strength of all lifting points and of all components shall be verified using a load factor γ of 2,0 to be applied to the static loads produced by the proposed erection method. This factor can be reduced to 1,5 if the operations are carefully controlled. As regard the strength factor of components, the respective values in Tables 18 to 21 corresponding to damage limits of components shall apply.

6.5.3 Construction stringing and sagging

6.5.3.1 Conductor tensions

The tensions shall be calculated at the minimum temperature allowed for stringing and sagging operations. It is recommended that in the calculation of loads on the structures, conductor tensions of at least twice the sagging tensions be used for conductors being moved and 1,5 times for all conductors in place.

6.5.3.2 Vertical loads

The extra load applied to the supports shall be calculated from the vertical exit angles of the conductor, with the conductor tensions given in 6.5.3.1. The loading shall be applied to the conductor attachment points or conductor stringing points (if different), and shall consider all possible conductor stringing sequences in any combination of load and no load at the several support points that represent the conductor stringing sequence.

6.5.3.3 Transverse loads

Angle supports shall be capable of resisting the transverse loads produced by the conductor tensions given in 6.5.3.1.

Although light winds can occur during construction and maintenance, their effect is neglected for these calculations.

6.5.3.4 Longitudinal (and vertical) loads on temporary dead-end supports

a) Longitudinal loads

   Supports used as dead-ends during stringing and sagging shall be capable of resisting longitudinal loads resulting from the sagging tensions given in 6.5.3.1 in any combination of load and no load at the several support points that represent the conductor stringing sequences.

b) Vertical loads

   If such supports are reinforced by temporary guys to obtain the required longitudinal strength, these guys will increase the vertical loads at the attachment points and shall be adequately pre-stressed if attached to a rigid support. It will therefore be necessary to check the tension in the guys and take account of the vertical loads applied to the attachment points.

NOTE Pre-stressing of guys is required because of differences in deformation of guys versus lattice crossarm when both are subjected to load.

6.5.3.5 Longitudinal loads on suspension supports

While the conductor is in the stringing sheaves, a longitudinal load shall be applied to the supports. This load is equal in value to the unit weight of the phase conductor, w (N/m), multiplied by the difference in elevation of the low points of adjacent spans (m). This load (in N) will be negligible and much less than the containment loads derived in 6.6.3 except for unusual spans, where it shall be verified that the support can resist at least twice this load.

In operations such as conductor tie-downs, loads are applied at all conductor points and shall be taken into account.

6.5.4 Maintenance loads

All conductor support points shall be able to resist at least twice the bare conductor vertical loads at sagging tensions.

Temporary lift or tension points, close to the normal attachment points of conductors and used for maintenance or live line operations, shall also be able to resist at least twice the bare conductor loads at sagging tensions.

A factor of 1,5 instead of 2 for the above loads can be used if the operations are carefully controlled.

Those responsible for maintenance shall specify lifting arrangements which will not overstress the support.

All structural members that may be required to support a lineman shall, by calculation, be able to support a 1 500 N load, applied vertically at their midpoint, conventionally combined with the stresses present during maintenance. These are usually based on still air at the minimum temperature assumed for maintenance operations.

6.6 Loads for failure containment (security requirements)

6.6.1 General

The objective of security measures is to minimize the probability of uncontrolled propagation of failures (cascades) which might otherwise extend well beyond the failed section, whatever the extent of the initial failure.

The security measures detailed below provide for minimum security requirements and a list of options which may be used whenever higher security is justified.

The loads prescribed in 6.6.3 provide conventional lattice structures with the means of minimizing the probability of cascade failures. These requirements are derived from experience on conventional lattice structures, but should also be applicable to other types of structures. Service experience using different types of structures or materials could dictate or require different or additional precautions that can be substituted to the above requirements.

The system stress under these loads shall not exceed the failure limit of its components described in 7.3.1.

6.6.2 Security requirements

Unless special limiting devices are used, the loadings specified in 6.6.3 shall be considered as minimum requirements applicable for most transmission lines.

In cases where increased security is justified or required (for example on important lines, river crossings or lines subjected to significant ice loads), additional measures or loadings can be used according to local practice and past experience. A list of such measures appears in 6.6.3.3.

It is noted that the security requirements described in the following clauses may not be effective in preventing cascading failures when a failure occurs in a long stretch of the line already loaded near to, or above, its limit. In such cases, a large number of towers located in this area may fail despite the presence of anti-cascading towers.

6.6.3 Security related loads – Torsional, longitudinal and additional security measures

6.6.3.1 Torsional load

At any one ground wire or phase conductor attachment point the relevant, if any, residual static load (RSL) resulting from the release of the tension of a whole phase conductor or of a ground wire in an adjacent span shall be applied. This RSL shall be considered at sagging temperatures without any wind or ice loads.

The *RSL* for suspension structures shall be calculated for average spans and at sagging tensions, allowance being made for the relaxation of the load resulting from any swing of the insulator strings assemblies, deflection or rotation of the structure, foundations, articulated crossarms or articulated supports, and the interaction with other phases conductors or wires that may influence this load.

The value of the *RSL* may be limited by special devices (slipping clamps, for example), in which case the minimum security requirements should be adjusted accordingly.

Coincident bare conductor loads at sagging tensions shall be applied at all other attachments points.

6.6.3.2 Longitudinal loads

Longitudinal loads shall be applied simultaneously at all attachment points. They shall be equal to the unbalanced loads produced by the tension of bare conductors in all spans in one direction from the structure and with a fictitious overload equal to the weight, w, of the conductors in all spans in the other direction. Average spans shall be considered with the bare conductors at sagging tension, and any appropriate relaxation effects, as mentioned in 6.6.3.1, shall be considered. See Table 15.

An alternative proposal would be to consider about 50 % of the sagging tension at each attachment point.

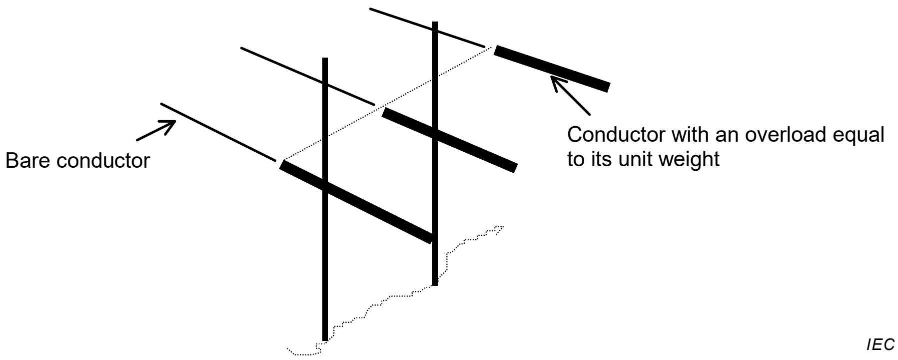

Figure 15 – Simulated longitudinal conductor load (case of a single circuit support)

6.6.3.3 Additional security measures

The designer can increase the line security by adopting some of the requirements listed in Table 13.

Table 13 – Additional security measures

Description of additional security measures	Comment
Increase the *RSL* by a factor of 1,5 to 1,8 at any one point	Lines where higher security is justified. This requirement will increase the probability of a suspension support to resist the dynamic load due to a broken conductor
Increase the number of torsional/flexural load points to either two phases or two ground wires where the residual static load (*RSL*) is applied	This option may apply to double or multi-circuit lines
Calculate the *RSL* for tensions higher than the every day load by using wind or ice load corresponding to a 3 year return period in conjunction with this loading case	Advisable for angle supports or lines subjected to severe climatic (icing) conditions
Insertion of anti-cascading support at intervals, typically every tenth support. These supports shall be designed for all broken conductors subjected to limit ice and/or wind loads	To be considered for important lines in icing areas

7 Strength of components and limit states

7.1 General

The purpose of this clause is to define limit states of line components strengths and their common statistical parameters.

When subjected to increasing loads, line components may exhibit a permanent deformation at some load level, particularly if the failure mode is ductile. This level is called the damage or serviceability limit state. If the load is further increased, failure (or rupture) of the component occurs at a level called the failure or ultimate limit state.

The transmission line is considered intact when its components are used at stresses below their damage limit. It is considered in a damaged state if its components have exceeded their damage limit state, but without exceeded any of the component's failure limit. Finally the line is considered to have failed if its components have reached their failure limit. The graphical interpretation is shown on Figure 16.

State of the system	Intact state	Damage state	Failure state
Strength limit of components		Damage limit (serviceability limit state)	Failure Limit (ultimate limit state)

IEC

Figure 16 – Diagram of limit states of line components

7.2 General formulas for the strength of components

7.2.1 General

With reference to Formulas (5) and (6):

$$\text{(effects of } Q_\text{T}) < \Phi_\text{N} \times \Phi_\text{S} \times \Phi_\text{Q} \times \Phi_\text{C} \times R_\text{c}$$

During design, each component shall satisfy load and strength requirements for reliability, security and safety conditions. In practice, two sets of formulas (reliability and safety) determine the damage characteristic strength that components are required to meet, and a third set of formulas (security) determines the failure characteristic strength that components are required to meet. In these formulas, the reliability conditions are normally expected to be the governing condition for the main components.

7.2.2 Values of strength factor Φ_N

Whenever a number N of components are expected to be subjected to the same limit load Q_T during a single occurrence of a climatic event, the characteristic strength of individual components shall be de-rated (multiplied) by a strength factor Φ_N, This factor depends on N and on characteristics of the strength distribution function (type and coefficient of variation v_R) of strength R.

In the absence of specific experience, the number N of supports subjected to the maximum load intensity during a single occurrence of climatic events can be derived from Table 14.

Table 14 – Number of supports subjected to maximum load intensity during any single occurrence of a climatic event

Loading	Flat to rolling terrain	Mountains
Maximum gust wind	1 (1 to 5)	1 (1 to 2)
Maximum ice	10 (5 to 50)	2 (1 to 10)
Maximum ice and wind	1 (1 to 5)	1 (1 to 5)
NOTE Values in brackets represent the typical range of supports based on a span of 400 m.		

The number of components other than supports can be directly derived from the number of supports thus selected.

Values of Φ_N are given in Table 15 and are based on a normal distribution function. In the same table, the values within brackets are based on the log-normal distribution function. Values derived from other distribution functions can be used if more representative of the strength of the component being designed.

In the case of high values of v_R and N (see the shaded cells with italic figures in Table 15), the value of Φ_N is very sensitive to the choice of the distribution function. Thus, engineering judgement and strength test results should be used in the selection of the appropriate distribution function. In Table 15 the values outside the shaded area are conservatively taken from the normal distribution curve. Should the strength distribution curve be known, CIGRE Technical Brochure 178 can be used to provide the specific values for the normal and log-normal distributions.

Practically, the lower value of Φ_N can be considered equal to 0,70 for most cases of N and COV of strength, while values from 0,95 to 1,0 can be used for weather events of limited spatial extent.

Table 15 – Strength factor Φ_N related to the number N of components or elements subjected to the critical load intensity

	Coefficient of variation of strength v_r						
N	**0,05**	**0,075**	**0,10**	**0,15**	**0,20**	**0,25**	**0,30**
1	1,00	1,00	1,00	1,00	1,00	1,00	1,00
2	0,98	0,98	0,97	0,94	0,91	0,87	0,84
5	0,96	0,94	0,92	0,85	0,80	*0,72 (0,83)*	*0,64 (0,80)*
10	0,94	0,92	0,89	0,81	*0,72 (0,82)*	*0,62 (0,77)*	*0,51 (0,73)*
20	0,93	0,90	0,85	*0,77 (0,83)*	*0,66 (0,77)*	*0,53 (0,73)*	*0,38 (0,68)*
40	0,92	0,87	0,83	*0,72 (0,80)*	*0,59 (0,74)*	*0,44 (0,69)*	*0,26 (0,64)*
80	0,91	0,86	*0,79 (0,84)*	*0,68 (0,77)*	*0,53 (0,71)*	*0,36 (0,65)*	*0,16 (0,60)*
160	0,90	0,85	*0,79 (0,83)*	*0,67 (0,76)*	*0,52 (0,69)*	*0,34 (0,62)*	*0,13 (0,57)*
The use of the shaded cells with italic figures is explained in 7.2.2.							

7.2.3 General basis for strength coordination

Transmission line components have different strength variations and responses to loading. When subjected to given loads, failure of components in series could occur whenever load exceeds strength in any component.

In order to decide on an appropriate strength coordination, the following criteria constitute a consensus within the overhead line industry:

a) The first component to fail should be chosen so as to introduce the least secondary load effect (dynamic or static) on other components in order to minimize the probability of a propagation of failure (cascading effect).

b) Repair time and costs following a failure should be kept to a minimum.

c) The first component to fail should ideally have a ratio of the damage limit to the failure limit near 1,0. It should be mentioned that it might be difficult to co-ordinate the strength of components when the least reliable one has a very large strength variation.

d) A low cost component in series with a high cost component should be designed to be at least as strong and reliable as the major component if the consequences of failure are as severe as failure of that major component. An exception of this criterion is when a component is purposely designed to act as a load limiting device. In such a case, its strength has to be well tuned with the component it is supposed to protect.

If line components such as suspension supports, tension supports, conductors, foundations and insulator strings are analysed using the above criteria, it can be concluded that:

- conductors should not be the weakest component because of a), b) and c);
- fittings because of d);
- tension support because of a) and b);
- and foundations because of b) and c).

The logical consequence of the considerations above is that the suspension supports should constitute the component with the lowest strength. When a line designed according to this rule is subjected to climatic loads exceeding design values the suspension supports would fail first. Despite the adoption of the above criteria there could be situations where conductors could fail because of its wires being severed during the collapse of a lattice support.

The above strength coordination can be applied to most lines. However there will be some situations where different criteria could be used and thus lead to another sequence of failure.

For example, special river crossing supports could be designed stronger than the conductors. In avalanche areas and/or in areas where construction of supports is very difficult, the conductor may also be chosen as the weakest component, provided that supports in this area are designed for the longitudinal forces resulting from the failure of the conductors. Otherwise the failure of conductors would very probably lead to the failure of adjacent supports. If a line section is made of dead-end towers capable of withstanding full tension of broken conductors, it is also possible in those cases to design conductors weaker than towers.

7.2.4 Strength factor Φ_S related to the coordination of strength

It is often cost-effective and a desirable feature to design some components to be more reliable than others in order to minimize the consequences (i.e. repair time, secondary failure, etc.) of a possible failure due to climatic event.

In order to achieve such strength coordination, a strength reduction factor Φ_{S2} is applied to the strength of components R_2 chosen to be more reliable while a factor Φ_{S1} = 1,0 is applied to the first component to fail. Factor Φ_{S2} depends on the coefficient of variation of both components and is given in Table 16. It is based on a confidence of 90 % that the second component R_2 will not fail before the first R_1. Thus, 90 % is the confidence level on the target sequence of failure.

Table 16 – Values of Φ_{S2}

		COV of R_1			
		0,05	**0,075**	**0,10**	**0,20**
COV of R_2	0,05 to 0,10	0,92	0,87	0,82	0,63
	0,10 to 0,40	0,94	0,89	0,86	0,66
NOTE In this table, R_2 is the component designed more reliable than R_1.					

Criteria for deciding on a preferred strength coordination are discussed in CIGRE Technical Brochure 178, and a usually accepted strength coordination is given in Table 17. This table provides first for the strength coordination between major components and, subsequently, provides for a subsequent coordination within the various elements of a major component.

Table 17 – Typical strength coordination of line components

	Major component	**Coordination within major components ***
Less reliable (first component likely to fail when limit loads are exceeded)	Suspension support	<u>Support</u>, foundations, interfaces
More reliable with 90 % confidence (less likely to fail first when limits loads are exceeded)	Tension support Dead-end support Conductors**	<u>Support</u>, foundations, interfaces <u>Support</u>, foundations, interfaces <u>Conductors</u>, insulators, interfaces
* Within each major component, the underlined component is the least reliable with 90 % confidence. ** With the strength limits specified in Table 20, conductors are usually the most reliable component of the line.		

7.2.5 Methods for calculating strength coordination factors Φ_S

In order to develop strength coordination factors Φ_S in the above Table 16 leading to the target strength coordination, two methods were considered:

- Use of different exclusion limits: For the weakest component, use limit loads in conjunction with 10 % exclusion limit (as suggested in this approach). The next weakest components will be designed with a lower exclusion limit (say 1 % to 2 %), corresponding to the same limit loads.
- Design for a target confidence level in the strength coordination: Strength coordination factors are established in such a way that the target strength coordination between two components, as mentioned in Table 17, will be reached with a high level of confidence (nearly 80 % to 90 %). However, due to the random nature of strength, it is theoretically impossible to guarantee with 100 % confidence that the planned coordination of strength will be met in all cases.

It is noted that strength coordination would be difficult and not cost efficient if a component with a large strength variation is chosen as the first component to fail. For example, as seen from Table 16, when v_{R1} = 0,20, the characteristic strength of the next strongest components would have to be selected such that it would meet the limit loads when multiplied by about 0,7.

Similarly, as seen from Table 16, it can be concluded that if suspension supports (usually v_R = 0,05 to 0,10) are designed as the weakest components, the characteristic strength of foundations (v_R is usually from 0,10 to 0,30) has to be multiplied by a factor of 0,83 to 0,93. In this case, there is 90 % confidence that foundations will not fail before the supported tower.

7.3 Data related to the calculation of components

7.3.1 Limit states for line components

Tables 18 to 21 specify damage limits and failure limits for line components with regard to the system. In the absence of relevant data, these values constitute acceptable design limits. If local data and national experience is available, it can be used to improve and complete the tables.

Table 18 – Damage and failure limits of supports

Supports			Damage limit	Failure limit
Type	**Material or elements**	**Loading mode**		
Lattice towers, self-supporting or guyed	All elements, except guys	Tension	Yield (elastic) stress	Ultimate (breaking) tensile stress
		Shear	90 % (elastic) shear stress	Shear (breaking) stress
		Compression (buckling)	Non-elastic deformation from $l/500$ to $l/100$	Collapse by instability
	Steel guys	Tension	Lowest value of: – yield stress (70 % to 75 % UTS) – deformation corresponding to 5 % reduction in tower strength – need to readjust tension	Ultimate tensile stress
Poles	Steel	Moments	1 % non-elastic deformation at the top, or elastic deformation that impairs clearances	Local buckling in compression or ultimate tensile stress in tension
		Compression (buckling)	Non elastic deformation from $l/500$ to $l/100$	Collapse by instability
	Wood	Moments	3 % non-elastic displacement at the top	Ultimate tensile stress
		Compression (buckling)	Non-elastic deformation from $l/500$ to $l/100$	Collapse by instability
	Concrete	Permanent or non- permanent loads	Crack opening after release of loads, or 0,5 % non-elastic deformation (The width of crack for concrete poles to be agreed upon).	Collapse of the pole

NOTE 1 The deformation of compression elements is the maximum deflection from the line joining end points. For elements subjected to moments, it is the displacement of the free end from the vertical.

NOTE 2 l is the free length of the element.

Table 19 – Damage and failure limits of foundations

<table>
<tr><th colspan="3">Foundations</th><th rowspan="2">Damage limit</th><th rowspan="2">Failure limit</th></tr>
<tr><th>Type</th><th>Support type</th><th>Statically determinate movement</th></tr>
<tr><td rowspan="4">Uplift</td><td rowspan="2">Guyed</td><td>Yes</td><td>Need to readjust tension in guys</td><td rowspan="4">Excessive out-of-plane uplift movement (plane formed by the other three foundations) in the order of 5 cm to 10 cm</td></tr>
<tr><td>No</td><td>5 % reduction in support strength</td></tr>
<tr><td rowspan="2">Self-supporting</td><td>Yes</td><td>1° (degree) rotation of the support</td></tr>
<tr><td>No</td><td>Differential vertical displacement equal to Y/300 to Y/500 with a maximum of 2 cm</td></tr>
<tr><td rowspan="2">Compression</td><td rowspan="2">All types</td><td>Yes</td><td>Displacement corresponding to a 5 % reduction in the support strength</td><td rowspan="2">Excessive out-of-plane settlement (plane formed by the other three foundations) (in the order of 5 cm to 10 cm)</td></tr>
<tr><td>No</td><td>Differential vertical displacement equal to Y/300 to Y/500 with a maximum of 2 cm</td></tr>
<tr><td rowspan="2">Moments (rotations)</td><td rowspan="2">Poles</td><td>Yes</td><td>2° (degree) rotation of the support</td><td rowspan="2">Excessive rotation in the order of 5° to 10°</td></tr>
<tr><td>No</td><td>Rotation corresponding to a 10 % increase in the total moment due to eccentricity</td></tr>
<tr><td colspan="5">NOTE 1 Takes into account the interaction between the support and its foundation.

NOTE 2 A statically determinate movement is one that does not induce internal efforts in the structure. For example the displacement of one foundation of a three-legged support is a statically determinate movement, while the displacement of a four-legged support is a statically indeterminate movement.

NOTE 3 Y is the horizontal distance between foundations.

NOTE 4 Some rigid foundations (e.g. pile) may require lower limits.</td></tr>
</table>

Table 20 – Damage and failure limits of conductors and ground wires

<table>
<tr><th>Conductors and ground wires</th><th>Damage limit</th><th>Failure limit</th></tr>
<tr><td>All types</td><td>Lowest of:
– vibration limit*, or
– the infringement of critical clearances defined by appropriate regulations, or
– 75 % of the characteristic strength or rated tensile strength (typical range in 70 % to 80 %)</td><td>Ultimate tensile stress (rupture)</td></tr>
<tr><td colspan="3">* A typical vibration limit is not to exceed a conductor parameter C (=H/w) = 2 000 m under initial unloaded conditions for average temperature of the coldest month. (Refer to CIGRE Technical Brochure 273 for more details).</td></tr>
</table>

Table 21 – Damage and failure limit of interface components

Type of interface components	Damage limit	Failure limit
Conductor joints – dead-end and junction fittings – suspension fittings	Unacceptable permanent deformation (including slippage)	Rupture
Insulators (porcelain and glass)	70 % strength rating or broken shed (glass only)	Rupture of pin, cap, cement or shed
Composite insulators	Typically 70 % of rating particularly if loads are not transient	Rupture
Fittings	Critical permanent deformation	Rupture of fittings or shear of bolts
NOTE 1 Normally, fittings are designed in a manner to reduce or eliminate wear. Should wear be expected because of point-to-point contact, it should be considered in the design. In such a case, the damage limit becomes 'exceeding the excepted wear'. NOTE 2 The critical permanent deformation is defined as the state where the fittings cannot be easily taken apart.		

7.3.2 Strength data of line components

For practical considerations, it is assumed that the normal density function is adequate for the statistical distribution of the strength of line components. As indicated earlier, log-normal density function can also be used to characterize strength variation, mainly for components with brittle behaviour or subjected to stringent quality control.

This assumption of normal density function is quite true for many line components, particularly those having a low coefficient of variation.

If no specific tests are available, the characteristic strength R_c will be found in ruling standards; R_c may be assumed to correspond to e = 10 %. Table 22 gives typical strength coefficient of variation v_R to be used as default value in the absence of relevant data.

If tests are available, $R_c = (10\ \%)R = (1 - u \times v_R)\ \overline{R}$; if R is assumed normally distributed, u = 1,28, or given in Table 23 for log-normal distribution function.

For additional information, refer to Annex A.

NOTE The value of u = 1,28 corresponds to a large number of samples. For fewer samples, different values derived from statistical properties of the normal distribution function can be used.

Table 22 – Default values for strength coefficients of variation (*COV*)

Component	*COV*
Conductors and ground wires (strength usually limited by joints)	0,03
Fittings	0,05
Insulators	0,05
Steel poles	0,05
Concrete poles	0,15
Wood poles	0,20
Lattice towers	0,10
Grouted rock anchors	0,10
Pile foundation	0,25
Foundation with undercut or machine-compacted backfill	0,20
Foundation with uncompacted backfill	0,30

Table 23 – *u* factors for log-normal distribution function for *e* = 10 %

COV	*u*
0,05	1,26
0,10	1,24
0,20	1,19
0,30	1,14
0,40	1,08

7.3.3 Support design strength

Supports shall be designed for a characteristic strength R_c equal to:

$$R_c \geq \frac{\text{Support design loads}}{\Phi_N \, \Phi_S \, \Phi_Q \, \Phi_C} \tag{27}$$

Support design loads comprise the dead loads and external loads.

Φ_N is selected according to 7.2.2.

Φ_S is derived from Table 16. It is equal to 1,0 if the support is selected as the least reliable component. Note that it may be advisable to design tower parts such as crossarms and ground wire peaks, with a sub-sequence of failure within the tower so that failure of these parts will not cause failure of main tower body.

Φ_Q for lattice towers,Table 24 gives recommended values for Φ_Q, to take into account the quality in calculation method, fabrication and erection. For other supports, coefficients Φ_Q of the same order can be estimated by view of local conditions.

Φ_C can be taken equal to 1,0, especially when the characteristic strength corresponds to a 10 % exclusion limit. If the exclusion limit varies greatly from 10 %, refer to CIGRE Technical Brochure 178 for possible adjustments.

Table 24 – Value of quality factor Φ_Q for lattice towers

Quality control	Φ_Q
Very good quality control such as involving third party inspection	1,00
Good quality control	0,95
Average quality control	0,90

Supports subjected to full scale (type) tests shall withstand loads equivalent to R_c. Tests shall conform to the latest version of IEC 60652.

7.3.4 Foundation design strength

The maximum reactions on foundations are obtained from the design of supports subjected to the loads defined in this standard using conventional methods of analysis and appropriate wind-weight span combinations, support legs and body extensions. The reactions thus obtained are considered to be the design loads on foundations. When foundation tests are required, these shall be performed in accordance with the latest version of IEC 61773.

The characteristic strength of foundations R_c, shall meet the following requirement:

$$R_c \geq \frac{\text{Foundation design loads}}{\Phi_N\, \Phi_S\, \Phi_Q\, \Phi_C}$$

Φ_N depends on the number of foundations subjected to maximum load intensity in a given storm event. For example, if N = 2, and COV = 0,20, Φ_N = 0,91 can be obtained from Table 15.

Φ_S can be obtained from Table 16, based on the expected strength COV. For default COV values, refer to Table 22.

If characteristic strength R_c is derived from tests typical of actual line construction, then Φ_Q = 1. However, if foundation tests were carried out in a controlled environment not typical of line construction, then it is suggested to consider Φ_Q = 0,9.

Φ_C can be taken equal to 1,0, specially when the characteristic strength corresponds to a 10 % exclusion limit. This is usually the case when R_c is deducted from foundation tests. In case the exclusion limit varies greatly from 10 %, refer to CIGRE Technical Brochure 178 for possible adjustments.

7.3.5 Conductor and ground wire design criteria

Conductors and ground wires are designed for the highest tension resulting from all loading cases applied to the line. This tension corresponds to the highest point in the span.

In this case, $\Phi_N = \Phi_S = \Phi_Q$ = 1,0 and the maximum conductor tension shall not exceed R_c as defined in Table 20.

When required, conductor tests shall comply with IEC 61089.

7.3.6 Insulator string design criteria

The calculation of the insulator strings is based on their relationship to the conductors to which they are attached. These are dealt with in the same way as for the support/foundation relationship. The critical design loading shall be derived from the maximum calculated conductor loading to which the insulator strings are attached. For suspension strings, the maximum loads are equal to the maximum resultant of the combined vertical, transverse and longitudinal loads applied to the attachment point of the insulator string. In the case of tension strings, it is equal to the maximum conductor tension.

Φ_N shall be derived in accordance withTable 15.

Φ_S = Φ_{S2} = 0,90 for all insulator strings, for which the COV generally remains under 7 % (see Table 16 and Table 21).

Φ_C = 1,0, and Φ_Q = 1,0 (unless poor quality material).

In addition to the above requirements, it is advisable, particularly for countries subjected to ice loads, to select the characteristic strengths of dead-end insulators at least as high as the characteristic strength R_c of attached conductors. Similarly, it is advisable to design the dead-end fittings to withstand, at failure, about 10 % to 15 % more than the conductor characteristic strength R_c. When required, tests for fittings shall comply with EC 61284.

Annex A
(informative)

Technical information – Strength of line components

A.1 Calculation of characteristic strength

The characteristic strength is defined as the strength guaranteed with a given probability.

If $\overline{R}$ is the mean strength of a component and v_R its coefficient of variation, then the characteristic strength R_c is given by formula:

$$R_c = \overline{R}\ (1 - u_e\ v_R) \qquad \text{(A.1)}$$

The value of v_R depends on the type of material and the fabrication practice (quality control). The variable factor u_e depends on the distribution function of the strength of the component and on the probability of exceeding the guaranteed strength, represented by the exclusion limit e.

The characteristic strength of line components in most countries corresponds to an exclusion limit (probability of not being achieved) lower than 10 % and usually in the order of 2 % to 5 %. Assuming a characteristic strength with a higher exclusion limit would produce a significant number of under-strength components and a very low exclusion limit may not be cost-effective, specially for components with high v_R. Thus, values from 2 % to 5 % correspond to a practical economic balance. If a normal distribution is assumed for strength R, u_e would thus vary between 1,60 and 2,10.

For example, in order to guarantee a minimum yield point of 300 MPa for a given grade of steel, a manufacturer, knowing that the coefficient of variation is 0,05, will generally produce a steel which has an mean strength of 300 / (1 – 2,10 × 0,05) = 340 MPa. The probability of not meeting the minimum strength (or the characteristic strength) is quite low and is in the order of 2 %. The same approach applies to insulators where it was found from compiled strength data that the characteristic strength corresponds to a very low exclusion limit (approximately 0,1 %).

Consequently, the exclusion limit of 10 % used in the reliability Formula (2) can be related to the characteristic value by means of:

$$(10\%)R = (1 - 1{,}28 v_R)\overline{R} = \frac{(1 - 1{,}28 v_R)R_c}{1 - u_e v_R} \qquad \text{(A.2)}$$

or

$$(10\ \%)R = \Phi_c\ R_c \qquad \text{(A.3)}$$

If the value of u_e is not known, it can be estimated according to Table A.1 which is based on the frequency of rejects calculated from the normal distribution.

Table A.1 – Values of u_e associated to exclusion limits

	Estimated frequency of rejects		
	Frequent	**Some**	**Rare**
Exclusion limit *e*	About 10 %	2 % to 5 %	< 2 %
u_e	1,28	1,6	2,1

Φ_c is a correction factor that can be applied to the characteristic strength R_c if there is enough evidence or data to warrant that the exclusion limit of R_c is different from 10 %.

$$\Phi_c = (1 - 1,28\ v_R) / (1 - u_e\ v_R) \tag{A.4}$$

In typical cases, Φ_c can be considered equal to 1,0 which should normally lead to a satisfying design reliability.

Annex B
(informative)

Formulas of curves and figures

B.1 General

The formulas provided hereinafter describe the curves in various figures of the standard. Many of the curves in this standard were originally developed in previous IEC/TC11 documents based on a combination of theoretical studies and experimental results that were fine tuned according to experience.

B.2 Formula for G_c – Figure 4

G_c = 0,291 4 × ln(z) + 1,046 8 (terrain type A)

G_c = 0,373 3 × ln(z) + 0,976 2 (terrain type B)

G_c = 0,493 6 × ln(z) + 0,912 4 (terrain type C)

G_c = 0,615 3 × ln(z) + 0,814 4 (terrain type D)

All above formulas apply to $z > 10$ m. In case $z < 10$, the value of G_c remains constant and equal to the value for z = 10 m.

B.3 Formula for G_L – Figure 5

$G_L = 4 \times 10^{-10} \times L^3 - 5 \times 10^{-7} \times L^2 - 10^{-4} \times L + 1{,}040\ 3$

If $L < 200$ m, $G_L = 1$

B.4 Formula for G_t – Figure 6

G_t = –0,000 2 × z^2 + 0,023 2 × z + 1,466 1 (terrain type A)

G_t = –0,000 2 × z^2 + 0,027 4 × z + 1,682 0 (terrain type B)

G_t = –0,000 2 × z^2 + 0,029 8 × z + 2,274 4 (terrain type C)

G_t = –0,000 2 × z^2 + 0,038 4 × z + 2,928 4 (terrain type D)

All above formulas apply to $z > 10$ m. In case $z < 10$, the value of G_t remains constant and equal to the value for z = 10 m.

B.5 Formula for C_{xt} – Figure 8 (flat-sided members)

$C_{xt1,2} = 4{,}172\ 7 \times \chi^2 - 6{,}168\ 1 \times \chi + 4{,}008\ 8$

B.6 Formula for C_{xt} – Figure 9 (round-sided members)

$C_{xt1,2}$ = 0,229 3 × χ^3 + 2,709 1 × χ^2 – 3,132 3 × χ + 2,200 2

B.7 Formulas for C_{xtc} – Figure 10

C_{xtc} = 1,2 when $\mathrm{Re} < 3 \times 10^5$

C_{xtc} = 0,75 when $\mathrm{Re} > 4,5 \times 10^5$

C_{xtc} = –1,109 8 × ln(Re) + 15,197, when $3 \times 10^5 < \mathrm{Re} < 4,5 \times 10^5$

Annex C
(informative)

Atmospheric icing

C.1 General

Atmospheric icing is a general term for a number of processes where water in various forms in the atmosphere freezes and adheres to objects exposed to the air. Generally, there are two types of icing which are named according to the main processes:

- precipitation icing, and
- in-cloud icing.

A third process, where water vapour is transformed directly into the ice phase and forms so-called "hoar frost", does not lead to significant loadings and is not considered further.

Precipitation icing occurs in several forms, among which the most important are

- freezing rain,
- wet snow accretion, and
- dry snow accretion.

C.2 Precipitation icing

C.2.1 Freezing rain

When raindrops or drizzle fall through a layer of cold air (sub-freezing temperatures), the water droplets become supercooled. Therefore, they are still in a liquid water phase and do not freeze before they hit the ground or any object in their path. The resulting accretion is a clear, solid ice called glaze, often with icicles. This accretion is very hard and strong, and therefore difficult to remove. The density is 800 kg/m^3 to 900 kg/m^3, depending on the content of air bubbles.

Freezing rain occurs mostly on wide plains or basins where relatively deep layers of cold air accumulate during spells of cold weather. When a low pressure system with a warm front with rain penetrates the area, the cold (and heavier) air may remain near the ground and thus favour the formation of glaze (temperature inversion). Such a situation may persist until the upper winds have managed to mix the cold surface layer of air with the warmer air aloft.

A similar situation may occur in the overlapping zones of cold air and warm air systems. The warmer air, often with precipitation, is lifted over the colder air and forms a frontal zone where precipitation is enhanced.

Usually there are only moderate winds during freezing rain events. Hence the amount of accreted ice depends on the precipitation rate and duration.

C.2.2 Wet snow

Normally, the temperature increases as snow flakes fall through the atmosphere. If the air temperature near the ground is above freezing, the snow flakes start to melt when passing the 0 °C isotherm and the flakes contain a mixture of ice and water (at 0 °C) until they eventually melt totally into raindrops if the warm layer is deep enough. As long as they are only partly melted they will adhere to objects in the airflow.

The density may vary widely (100 kg/m^3 to 800 kg/m^3), but mostly from about 400 kg/m^3 to 600 kg/m^3. The density and intensity of accreted wet snow depends on the precipitation rate, wind speed and temperature. If the temperature drops below 0 °C after the accretion, the layer will freeze into a hard and dense layer with strong adhesion to the object.

Wet snow may also freeze on objects in colder air near the ground as in the case of freezing rain.

NOTE The density of wet snow accretion (type of precipitation icing) usually increases with wind speed, thus resulting in a smaller area exposed to wind pressure. In such case, it is possible that the resulting forces on the cables subjected to increased wind speed (height variation of Figure 12) may be less critical than at lower wind speed at 10 m reference height.

C.3 Dry ice

The ice growth is said to be dry when the available heat transfer rate away from the object is greater than the release of the latent heat of fusion. The density of the accretion is a function of the flux of water to the surface and the temperature of the layer. The resulting accreted ice is called soft or hard rime according to the density. A typical density for soft rime is 300 kg/m^3 and 700 kg/m^3 for hard rime.

C.4 In-cloud icing

In-cloud icing is a process whereby supercooled water droplets in a cloud or fog, freeze immediately upon impact on objects in the air flow, e.g. overhead lines in mountains above the cloud base.

The ice growth is said to be dry when the transfer of potential transfer of heat away from the object is greater than the release of the latent heat of fusion. The resulting accreted ice is called soft or hard rime according to its density which is typically 300 kg/m^3 for soft rime and 700 kg/m^3 for hard rime.

The ice growth is said to be wet when the heat transfer rate is less than the rate of latent heat release. Then the growth takes place at the melting point, resulting in a water film on the surface. The accreted ice is called glaze with a density of approximately 900 kg/m^3.

The icing rate varies mainly as a result of the following:

- liquid water content of the air;
- median volume droplet size of the spectrum;
- wind speed;
- temperature;
- dimensions of the iced object.

At temperatures below –10 °C, the water content of the air becomes smaller and less icing occurs. However, 8 kg/m was recorded in Switzerland with a temperature below –20 °C and strong winds.

Under the same conditions, the ice accretion rate will be greater for a small object than for a large one. Thus, heavy ice loadings are relatively more important for conductors than solid supports.

It should be noted that the heaviest in-cloud icing for specific locations, e.g. coastal mountains is usually due to a combination of wet-snow and hard rime.

C.5 Physical properties of ice

The physical properties of atmospheric ice may vary within rather wide limits. Typical properties are listed in Table C.1.

Table C.1 – Physical properties of ice

Type of ice	Density kg/m³	Adhesion	Appearance		Cohesion
			Colour	Shape	
Glaze ice	700 to 900	Strong	Transparent	Cylindrical icicles	Strong
Wet snow	300 to 700	Medium	White	Cylindrical	Medium to strong
Hard rime	700 to 900	Strong	Opaque to transparent	Eccentric pennants into wind	Very strong
Soft rime	200 to 600	Medium	White	Eccentric pennants into wind	Low to medium

C.6 Meteorological parameters controlling ice accretion

Table C.2 gives typical values of parameters that control the ice accretion.

Table C.2 – Meteorological parameters controlling ice accretion

Type of ice	Air temperature t °C	Mean wind speed V m/s	Droplet size	Liquid water content	Typical storm duration
Glaze ice	$-10 < t < 0$	Any	Large	Medium	Hours
Wet snow	$0 < t < 3$	Any	Flakes	Very high	Hours
Hard rime	$-10 < t < 1$	$10 < V$	Medium	Medium to high	Days
Soft rime	$-20 < t < 1$	$V < 10$	Small	Low	Days

The transition between soft rime, hard rime and glaze for in-cloud icing is mainly a function of air temperature and wind speed as shown in Figure C.1. However, the curves in Figure C.1 shift to the right with increasing liquid water content and with decreasing object size.

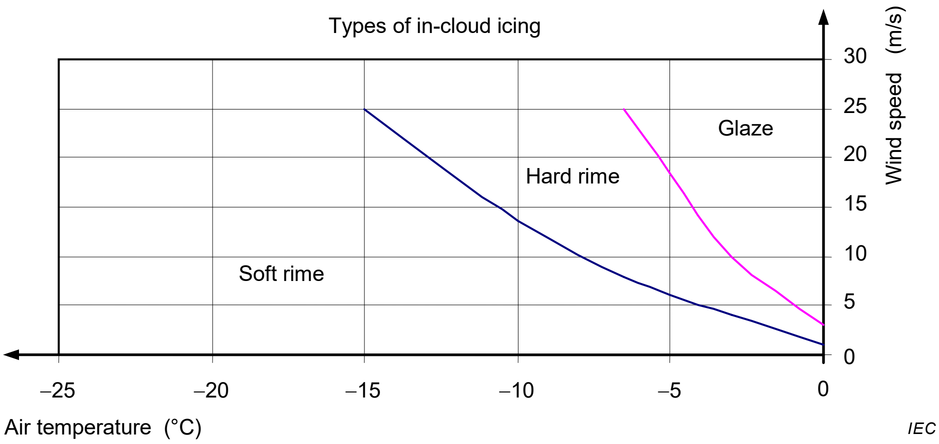

Figure C.1 – Type of accreted in-cloud icing as a function of wind speed and temperature

C.7 Terrain influences

C.7.1 In-cloud icing

The regional and local topography (large and medium scale) modifies the vertical motions of the atmosphere and hence the cloud structure and icing. Coastal mountains along the windward side of the continents act to force moist air upwards, leading to a cooling of the air with condensation of water vapour and droplet growth, eventually with precipitation. The most severe in-cloud icing occurs above the condensation level and the freezing level on freely exposed heights, where mountain valleys force moist air through passes and thus both lift the air and strengthen the wind.

On the leeward side of the mountains, the descent of an air mass results in internal heating of the air and evaporation of droplets, eventually with a total dissolution of clouds. A local shelter of hills not more than 50 m higher on the windward side may give a significant reduction in ice loadings. For this reason, routes in high mountains may very well be suited for overhead lines, provided they are sheltered against icing wind directions.

C.7.2 Precipitation icing

In general, precipitation icing may occur at any altitude. However, the probability of precipitation icing is generally greater in the valley basin than half way up the valley sides because of higher occurrence of cold air. Both freezing rain and wet snow may occur on large plains.

The greatest amounts of wet snow may be formed where the transverse wind component is strongest. Hence, an overhead line along a valley has fewer accretions than a line crossing the valley.

However, smooth hills or mountains transverse to the wind may cause the wind to strengthen on the leeward side, especially if there are no obstacles to such a flow on this side. Combined with wet snow, such hillsides may have significant failure probabilities for high ice loads combined with high wind velocities.

C.8 Guidelines for the implementation of an ice observation program

At the current time of writing, there seems to be practically no indirect way of getting proper data for design, although significant efforts have been made to develop models based on meteorological data and the collection of general experience from the areas of interest. As for any other type of structure depending on extreme values of wind speed, snow depth or temperature, the transmission line designer needs data and measurements of the most critical design parameters. Therefore, a program for collecting field data is strongly recommended, both from existing overhead lines and from especially designed devices.

Field ice data can be obtained by the following means:

a) Direct measurements of icing thickness or weight of samples taken from structures and line conductors. Ice samples fallen on the ground can be used, if consideration is given to the shape of initial ice accretion on conductors and to the fact that fallen pieces may represent only a fraction of the ice coating on the conductor.

b) Measurement on devices that simulate ice accretion on conductors. Devices currently used in some countries consist of simple tubes, rods or cable assemblies, 2 m to 5 m above ground level in order to facilitate measurement.

c) Estimation of icing using conductor tension or the vertical component of weight at the attachment point at the support.

d) Estimation of icing using the conductor sag.

Ice loading data are important not only to establish design load criteria and their associated failure probabilities, but can also be useful in the planning stages of the transmission network and route selection of transmission lines.

Since very few countries have data on ice loadings at their disposal, and considering that it takes at least 10 years of field observation to acquire such a data base, it is strongly recommended that any utility planning a major line project should undertake an ice observation programme without delay. Very often it will be possible to obtain the collaboration of the national weather services for the operation of instruments placed in standard meteorological stations.

However, any source of available and useful information should be collected and combined systematically in order to reduce, as much as possible, the time and cost of field measurements.

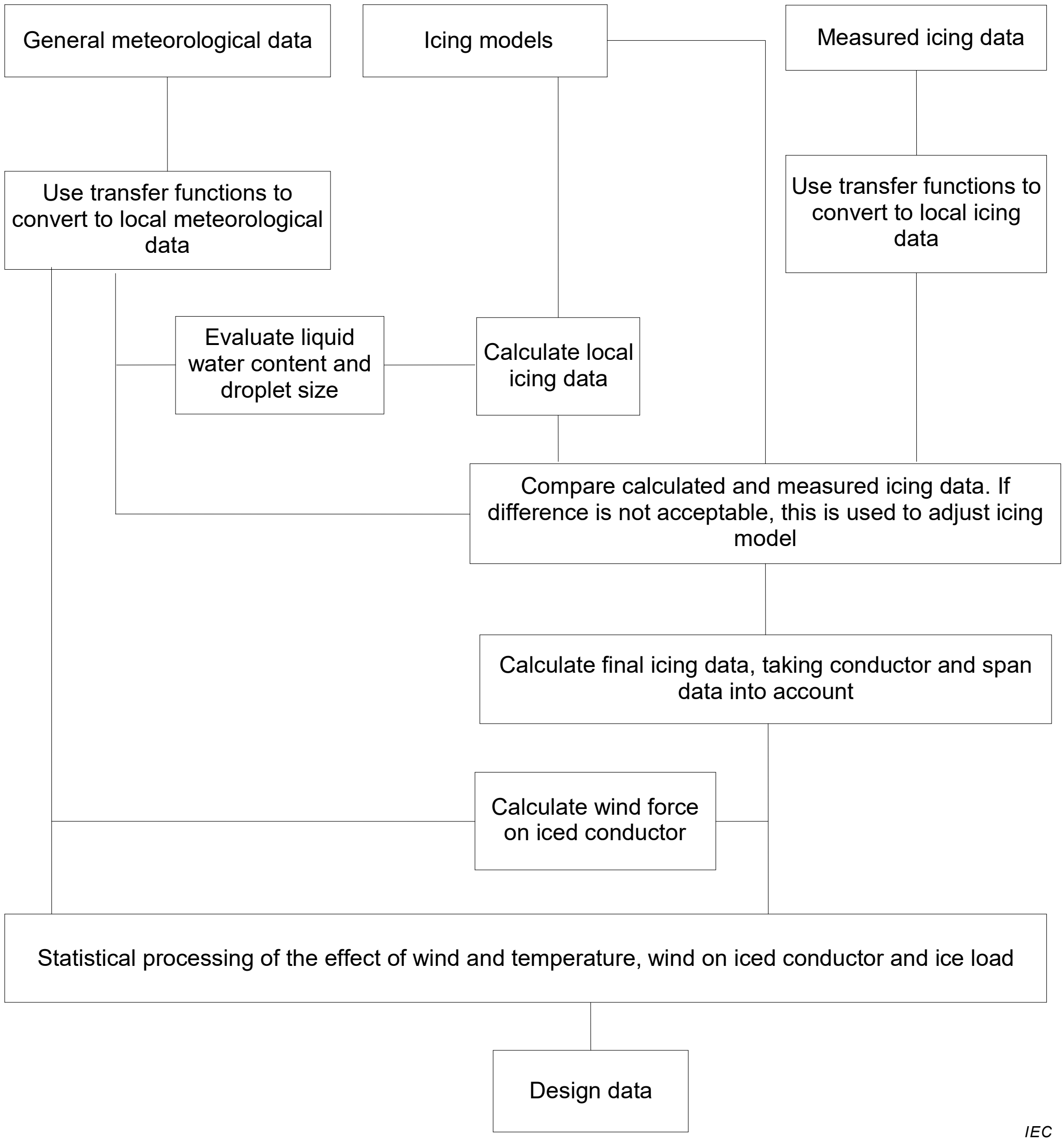

Figure C.2 – Strategy flow chart for utilizing meteorological data, icing models and field measurements of ice loads

IEC 61774 is a comprehensive standard covering all needs for meteorological data needed for overhead line design, and where measurement of icing is a significant part. Figure C.2 is taken from that standard. It demonstrates the strategy recommended in order to optimize the information that can be extracted from general meteorological data, icing models and separate measurements of ice loadings. Further guidelines on the selection of measuring sites, measuring devices as well as other instrumentation are also included in this standard.

Finally, it is recommended that the implementation of any major observation programme, as well as the analysis of the meteorological data, is conducted under the supervision of a professional meteorologist.

C.9 Ice data

C.9.1 Influence of height and conductor diameter

The diameter factor K_d, for conductor diameter d, is given in 6.3.4.1 as well as the height factor K_h , for height z above ground. They can be approximated by the following formulae.

For in-cloud icing:

$$K_d \sim 0{,}15\ d/30 + 0{,}85 \qquad \text{(C.1)}$$

NOTE The variation of in-cloud icing with height is very dependent on local topographical and climate conditions. Thus a specific climatic study is suggested to assess this variation for a line exposed to in-cloud icing.

and for precipitation icing

$$K_d \sim 0{,}35\ d/30 + 0{,}65 \qquad \text{(C.2)}$$

$$K_h = 0{,}075 \times z \ / \ 10 + 0{,}925 \qquad \text{(C.3)}$$

The above value of K_h for precipitation icing has been obtained using a simple icing model with a wind speed of about 25 km/h at 10 m and droplets fall speed of about 5 m/s. In case if conductor diameter d is less than 30 mm, K_d can be considered equal to 1,0 for both types of icing.

C.9.2 The effect of icing on structures

Ice accretion on structures increases their vertical loads on the structure and may control the design of foundations and some support members.

The weight of ice can be calculated using the geometry of the support members and the relevant thickness of ice accretion. Alternatively, an approximation can be derived from the following Table C.3:

Table C.3 – Approximate values of ice weights on lattice structures

Ice thickness (mm)	15	25	30	35	40	45	50
Ratio of weight of ice to structure weight	0,57	1,00	1,23	1,48	1,73	2,00	2,28

C.10 Combined wind and ice loadings

C.10.1 Combined probabilities

The action of wind on ice-covered conductors involves at least three variables: wind associated with icing situations, ice weight and ice shape. These combined effects can result in both transversal and vertical loads. Direct measurements of these loads should, ideally, be the best approach but due to the difficulties and cost involved, such measurements are scarce and are not usually available.

Since it is possible to obtain independent observations of wind speed, ice weight and ice shape, it is proposed to combine these variables in such a way that the resulting load will have at least approximately the same return periods T as those adopted for each reliability level.

Combining the probabilities of correlated variables would, however, require the knowledge of the various interacting effects of these variables on the loadings. Assuming that maximum loads are most likely to be related to maximum values of individual variables (wind speed, ice weight and ice shape), a simplified method is proposed. A low probability value of a variable (index L) is combined with high probability (index H) values of the other two variables, as shown in Table C.4. In this method, a certain degree of independence between the different variables is accepted.

Table C.4 – Combined wind and ice loading conditions

Loading conditions	Ice weight	Wind speed	Effective drag coefficient	Density
Condition 1	g_{L}	V_{iH}	C_{iH}	δ_1
Condition 2	g_{H}	V_{iL}	C_{iH}	δ_1
Condition 3	g_{H}	V_{iH}	C_{iL}	δ_2

The high probability is considered to be the average of extreme yearly values, while the low probability of the variable is the one corresponding to a return period T.

C.10.2 Drag coefficients of ice-covered conductors

Field measurement is the best approach for the determination of the drag or lift coefficients of ice-covered conductors. However, at the current time of writing, very few such measurements exist. As a result, statistical distributions of drag or lift coefficients are not yet known.

As long as statistical data on the effective drag coefficients and densities are not available, it is suggested, in the absence of other experimental values, that the values given in Table C.5 should be used.

Table C.5 – Drag coefficients and density of ice-covered conductors

	Wet snow	Soft rime	Hard rime	Glaze ice
Effective drag coefficient C_{IH}	1,0	1,2	1,1	1,0
Density δ_1 (kg/m^3)	700	600	900	900
Effective drag coefficient C_{IL}	1,4	1,7	1,5	1,4
Density δ_2 (kg/m^3)	300	400	700	900

Annex D
(informative)

Application of statistical distribution functions to load and strength of overhead lines

An analysis of meteorological data has shown that the distribution of annual maximum wind velocities or ice loads and ice thicknesses can be represented, with good approximation, by an extreme value distribution law (Gumbel Type I).

The basic formula for the Gumbel Type I cumulative distribution function has the form:

$$F(x) = \exp\{-\exp[-a \times (x-u)]\} \quad \text{(D.1)}$$

where $a = C_1/\sigma$ and $u = \bar{x} - C_2/a$ (D.2)

This formula expresses the probability $F(x)$ that a random value will be less than a value x in a distribution with a mean value $\bar{x}$ and a standard deviation σ.

The parameters C_1 and C_2 depend on the number of years (n) with observations and are given in Table D.1. For calculation of C_1 and C_2, see Table D.1.

Table D.1 – Parameters C_1 and C_2 of Gumbel distribution

N	C_1	C_2	C_2/C_1
10	0,949 63	0,495 21	0,521 48
15	1,020 57	0,512 84	0,502 50
20	1,062 82	0,523 55	0,492 60
25	1,091 45	0,530 86	0,486 39
30	1,112 37	0,536 22	0,482 05
35	1,128 47	0,540 34	0,478 82
40	1,141 31	0,543 62	0,476 31
45	1,151 84	0,546 30	0,474 28
50	1,160 66	0,548 54	0,472 61
∞	1,282 55= $\pi/\sqrt{6}$	0,577 22=Euler constant	0,450 05

The general form of Formula (D.1) thus becomes:

$$F(x) = \exp\left\{-\exp\left[-\frac{C_1}{\sigma}\left(x - \bar{x} + \frac{C_2 \times \sigma}{C_1}\right)\right]\right\} \quad \text{(D.3)}$$

and in the case where $n \approx \infty$

$$F(x) = \exp\left\{-\exp\left[-\pi\left(x - \bar{x} + 0{,}45\sigma\right)/\left(\sigma \times \sqrt{6}\right)\right]\right\} \quad \text{(D.4)}$$

Hence, the probability $P(x)$ that the observed value will be higher than x during one year is:

$$P(x) = 1-\exp\left\{-\exp\left[-\pi\left(x-\bar{x}+0{,}45\sigma\right)/\left(\sigma\cdot\sqrt{6}\right)\right]\right\} \quad \text{(D.5)}$$

As a simplification the return period T of the value x is given by:

$$T = \frac{1}{P(x)} \quad \text{(D.6)}$$

By rearranging the Formulae (D.3) and (D.6), the following is obtained:

$$x = \bar{x} - \frac{C_2\sigma}{C_1} - \frac{\sigma}{C_1}\left[\ln\left(-\ln\left(1-1/T\right)\right)\right] \quad \text{(D.7)}$$

The inclusion of C_1 and C_2 values in formula (D.3) will result in higher predictions if the number of years of data is reduced. However, it is noted that some countries use formulas (D.4) or (D.5) (i.e. C_1=1,282 55 and C_2/C_1=0,450 05) without taking into account the effect of number of years of data.

Table D.2 – Ratios of x / $\bar{x}$ for a Gumbel distribution function, T return period in years of loading event, n number of years with observations, v_x coefficient of variation

COV	*T*	Reliability level 1 50 years						Reliability level 2 150 years						Reliability level 3 500 years					
v_x	*n*	**10**	**15**	**20**	**25**	**50**	**∞**	**10**	**15**	**20**	**25**	**50**	**∞**	**10**	**15**	**20**	**25**	**50**	**∞**
0,05		1,18	1,17	1,16	1,15	1,14	1,13	1,24	1,22	1,21	1,21	1,19	1,17	1,30	1,28	1,27	1,26	1,24	1,22
0,075		1,27	1,25	1,24	1,23	1,22	1,19	1,36	1,33	1,32	1,31	1,29	1,26	1,45	1,42	1,40	1,39	1,37	1,33
0,10		1,36	1,33	1,32	1,31	1,29	1,26	1,48	1,44	1,42	1,41	1,38	1,36	1,60	1,56	1,54	1,52	1,49	1,44
0,12		1,43	1,40	1,38	1,37	1,35	1,31	1,57	1,53	1,51	1,49	1,46	1,41	1,72	1,67	1,64	1,62	1,59	1,53
0,15		1,54	1,50	1,48	1,46	1,43	1,39	1,71	1,66	1,63	1,62	1,58	1,52	1,90	1,84	1,80	1,78	1,73	1,66
0,16		1,57	1,53	1,51	1,49	1,46	1,41	1,76	1,70	1,68	1,66	1,61	1,55	1,96	1,89	1,86	1,83	1,78	1,70
0,20		1,72	1,66	1,64	1,62	1,58	1,52	1,95	1,88	1,84	1,82	1,77	1,69	2,20	2,12	2,07	2,04	1,98	1,88
0,25		1,90	1,83	1,79	1,77	1,72	1,65	2,19	2,10	2,05	2,03	1,96	1,86	2,51	2,40	2,34	2,30	2,22	2,10
0,30		2,08	2,00	1,95	1,93	1,87	1,78	2,43	2,32	2,27	2,23	2,15	2,04	2,81	2,68	2,61	2,56	2,46	2,32
0,35		2,26	2,16	2,11	2,06	2,01	1,91	2,66	2,54	2,48	2,44	2,34	2,21	3,11	2,96	2,87	2,82	2,71	2,54
0,40		2,43	2,33	2,27	2,24	2,16	2,04	2,90	2,76	2,69	2,64	2,54	2,36	3,41	3,23	3,14	3,08	2,95	2,76
0,45		2,61	2,49	2,43	2,39	2,30	2,17	3,14	2,98	2,90	2,85	2,73	2,55	3,71	3,51	3,41	3,34	3,20	2,98
0,50		2,79	2,66	2,59	2,54	2,44	2,30	3,38	3,20	3,11	3,05	2,92	2,73	4,01	3,79	3,68	3,60	3,44	3,20
0,55		2,97	2,83	2,75	2,70	2,59	2,43	3,61	3,42	3,32	3,26	3,11	2,90	4,31	4,07	3,94	3,86	3,68	3,42
0,60		3,15	2,99	2,91	2,85	2,73	2,56	3,85	3,64	3,53	3,46	3,30	3,07	4,61	4,35	4,21	4,12	3,93	3,64
0,65		3,33	3,16	3,07	3,01	2,88	2,68	4,09	3,86	3,74	3,67	3,50	3,25	4,91	4,63	4,48	4,38	4,17	3,86

Annex E
(informative)

Effect of span variation on load-strength relationship – Calculation of span use factor

E.1 General

Assuming F is the force resulting from climatic actions applied to the maximum span L_{max}, then the force on a support with a span L_{i} is in linear systems equal to $F \times L_{\text{i}}/L_{\text{max}}$. In the case of wind loads and a large difference between L_{i} and L_{max} there is small non-linearity introduced by the variation of gust factor with spans. This has little influence on the reliability because the latter is controlled by spans near maximum values where the span/load relation can be assumed to be linear. The ratio of $L_{\text{i}}/L_{\text{max}}$ is a random variable called use factor U. The use factor has an upper bound of 1,0 and a lower bound typically equal to 0,4. From the analysis of lines designed according to limit load concept, it has been found that the use factor can be assumed to have a Beta distribution function.

The use factor depends mainly on three variables: the number of types of suspension supports available for spotting, the category of the terrain and the constraints on support locations. For example, if every support in a line is custom-designed for the exact span at each location, the use factor will be equal to 1,0. While if only one suspension support type is used in a line located in mountainous terrain, the average use factor will be significantly less than 1, typically 0,60 to 0,75.

The use factor variation was found to have predictable patterns and statistical parameters $\overline{U}$ and σ_{U} could be known with sufficient accuracy if the number of suspension support types and spans, terrain and spotting constraints were known.

In CIGRE Technical Brochure 178, typical mean values $\overline{U}$ and standard deviation σ_u are given. Note that $\overline{U}$ can be derived from the design criteria of tangent supports if the average span of the transmission line is known, because of the following relation:

Average span = Average wind span = Average weight span

Thus, the average wind use factor $\overline{U}_{\text{wind}}$ can be calculated from:

$$\overline{U}_{\text{wind}} = \frac{\text{Average wind span}}{\text{Design wind span}} = \frac{\text{Average span}}{\text{Design wind span}} \tag{E.1}$$

Similarly,

$$\overline{U}_{\text{weight}} = \frac{\text{Average weight span}}{\text{Design weight span}} = \frac{\text{Average span}}{\text{Design weight span}} \tag{E.2}$$

As regards the values of σ_{u}, they can be deduced from a statistical analysis of span variations in the line.

E.2 Effect of use factor on load reduction and its calculation

As discussed in 5.2.3, when all supports are not used with their maximum spans, this contributes to an increase in reliability.

When the designer aims to design for a target reliability, he can, provided that sufficient data on span variation is available, reduce the design loads on supports by a factor γ_u <1 and achieve more economical lines. In case a factor $\gamma_u = 1$ is used for the design of structures, the overall reliability will be higher than expected.

Values of γ_u were computed for various cases of terrain types and suspension supports families and the recommended[3] values of γ_u are given in Table E.1 below.

Table E.1 – Use factor coefficient γ_u

$\overline{U}$	Recommended value of γ_u
0,95	0,95
0,85	0,90
0,80	0,88
0,75	0,83

As discussed earlier, span dispersion can be viewed as equivalent to either a strength increase by a factor of $1/\gamma_u$, or a decrease in applied loads, which are affected by the span lengths, by γ_u. The values of γ_u, if taken equal to 1,0 during the design of supports, will lead to a reliability higher than expected.

In cases where the considered supports will be used in different transmission lines where use factor coefficients can vary, it could be acceptable to use as a conservative value, the highest γ_u, or even $\gamma_u = 1,0$ which will lead to some over design. In the latter case, it is advisable to recheck the coordination of strength that may now be altered.

3 Detailed derivation of these factors can be found in Chapter B.3 of CIGRE Technical Brochure 178.

Annex F
(normative)

Conductor tension limits

F.1 General

In the process of designing overhead lines, tension limits of conductors are traditionally specified as a percentage of the conductor ultimate tensile strength (UTS). Invariably, these limits were intended to protect conductors from damaging aeolian vibrations and to provide enough safety margins that prevent conductors from breaking or overstretching under design ice and/or wind loads.

The percentages of UTS limits vary between countries and even between different areas in some countries. The following limits are typical of many practices in various jurisdictions.

a) Maximum conductor tension $<$ 50 % to 75 % of UTS under design wind/ice loads/temperature. The purpose of this limit is to prevent conductor failure under extreme weather conditions as well as the occurrence of excessive plastic stretch when conductors are taken to stresses in the range of 80 % to 95 % of their UTS.

b) Final bare conductor tension (without ice or wind loads) $<$ 18 % to 25 % UTS after the conductor has been subjected to long term creep or permanent plastic deformation due to load. This and the following condition were meant to protect the conductor from fatigue failure due to aeolian vibration. The percentages of 18 % to 25 % UTS were based on the assumption that conductors would be equipped with vibration dampers.

c) Initial bare conductor tension $<$ 20 % to 35 % when a new conductor is strung and sagged (prior to any long term creep). This condition is similar to the previous one, except that it tends to protect the conductors from fatigue damage at the initial stage, a few months following the construction, when the conductor tension is high and long term creep has not yet occurred.

The above limits were found to be acceptable in many countries and lines using mostly Aluminium Conductors Steel Reinforced (ACSR or A1/S1A according to IEC 61089) with steel to aluminium alloy ratio less than 13 % and in the case of all aluminium alloy conductors.
In the past decades, many other conductor types and strandings were used and it was found that the above tension limits may not provide enough vibration fatigue protection for mixed conductors having a high steel ratio or for all aluminium alloy conductors.

There is now a consensus within the experts that the tension limit to control aeolian vibrations should not be based on a percentage of UTS as in the past practices, but rather on the value of the conductor catenary parameter C (m). This parameter is equal to the horizontal conductor tension (N) divided by the unit weight of the conductor (N/m).

NOTE In case the conductor is loaded with wind and/or ice, the unit weight is replaced by the resultant load per unit length of the conductor.

This standard now recommends the use of mainly two conductor tension limits: the first limit applies to unloaded conductors and aims to prevent conductor fatigue due to aeolian vibration, while the second limit aims to prevent excessive conductor stretch due to the plastic deformation of the ice/wind loaded conductor.

Since the conductor tension varies with time due to long term creep as well as due to the plastic stretch of the conductors related to ice/wind loads, there is also a debate about which conductor condition or catenary parameter should be used to control damaging aeolian vibrations. The recent study given in CIGRE Technical Brochure 273 proposes to use the initial conductor tension at average temperatures of the coldest month as a basis to establish the limits of the catenary parameter C (equal to the horizontal tension divided by the unit conductor load per metre).

Based on CIGRE Technical Brochure 273, limiting the initial conductor tension at average temperatures of the coldest month to a maximum value of catenary parameter C of 2 000 m should be sufficient to reduce the chances of conductor fatigue due to aeolian vibrations when vibration dampers are used. Without vibration dampers, the same limit drops to about 1 000 to 1 400 m depending on terrain types. Since the CIGRE Technical Brochure does propose any limit for the maximum conductor tension, it is recommended to adopt the limit of 70 % to 80 % UTS proposed in Table 20. In such a case, the tension should be calculated under the limit design loads used for the overhead line.

NOTE In case of design according to working loads method where a safety factor is imposed by the relevant standard/practice, it has to be checked that the conductor tension under the combination of working loads increased by the safety or overload factor do not exceed to 70 % to 80 % UTS limit.

F.2 Limits for lines with short spans

In addition to the above, it should be recognized that lines with short spans (less than 100 m to 150 m) do not benefit significantly from high conductor tensions or catenary parameters as in the case of long spans ($>$ 300 m). The Table F.1 below provides the sags for a curlew conductor and two span cases of 400 m and 100 m, both strung at C values from 1 000 m to 2 000 m.

Table F.1 – Variation of conductor sag with catenary parameter C

Catenary parameter in m at –5 °C, initial conditions	Sag at 65 °C for 400 m span	Sag at 65 °C for 100 m span
2 000	13,90	1,86
1 800	14,59	1,96
1 500	16,58	2,13
1 200	19,29	2,32
1 000	22,34	2,48

As seen from the table above, the sag of the short span (e.g. distribution line) will increase only by 0,6 m for a reduction of C from 2 000 m to 1 000 m, while the same reduction in C value will increase the sag of the long span (e.g. transmission lines) by 8,5 m.

While the limit of $C <$ 2 000 m stated above provides a safe limit for vibration purposes, it may not be economical to use this limit in the case of lines having spans less than 100 m. It has to be remembered that increasing conductor tension, although it reduces conductor sag, it will also increase loads on angle supports.

F.3 Recommended conductor limit tensions

F.3.1 Initial tension limit

The initial tension limit at average temperatures during the coldest month (usually January) should not exceed a catenary parameter of 2 000 m for single conductor spans properly equipped with vibration dampers. In the case of bundled conductors, the limit of C = 2 000 m can be increased to 2 200 m or more if supported by a vibration study and/or an experimental verification.

NOTE This limit does not apply to special conductors such at self-damping conductors where different limits may be used in accordance with past experience and appropriate studies nor to large river crossings where higher conductor tensions may lead to significant economical benefits. In such cases, special studies and vibration control devices are suggested in order to mitigate the risk of fatigue damage of conductors.

F.3.2 Maximum final tension limit

The final tension limit after creep or permanent stretch due to ice/wind loads should not exceed the damage limit of the conductor, i.e. 70 % to 80 % UTS.

F.4 Benefits from reducing conductor tensions

For economical reasons, consideration can be given to reducing the limit of 2 000 m when spans are less than 400 m since such reduction will not impact much the conductor sag, but will provide additional safety for fatigue damage to conductors due to aeolian vibration as well as reducing loads on angles supports. Recommended reductions of the catenary parameter of 2 000 m are specified in Table F.2.

Table F.2 – Conductor tensioning – recommended catenary parameter limits

Span length m	**Recommended initial maximum catenary parameter*** m	**Corresponding % UTS for a 54/7 A1/S1A (ACSR) conductor**	**Corresponding % UTS for 37 A1 (ASC) conductor**	**Corresponding % UTS for 37 A2 (AASC) conductor**	**Corresponding %UTS for 45/7 A1/S1A (ACSR) conductor**
≥ 400	2 000	24	33	18	26
350 to 400	1 900	22	31	17	25
300 to 350	1 800	21	30	16	23
250 to 300	1 700	20	28	15	22
200 to 250	1 600	19	26	14	21
150 to 200	1 500	18	25	13	20
100 to 150	1 400	16	23	12	18
< 100	1 300	15	21	11	17

* The reduction of the catenary parameter in relation to the span is based on the principle that lower tensions are safer than higher ones from the point of view of aeolian vibrations. At the same time, this reduction will not affect the sags significantly, but will reduce loads on angle supports.

Annex G
(informative)

Methods of calculation for wind speed up effects due to local topography

G.1 Application

Overhead lines located in the proximity of topographical features such as hills, ridges, escarpments or spurs can experience significantly increased wind speeds due to topographical effects, particularly if they are close to peaks.

In general, wind speed up due to local topography needs only be considered where the site lies within the lesser of 3 km or 100 times the feature height (H) in a downwind direction from features defined as significant below.

Topographical features shall be considered to be significant if they have both the following characteristics:

a) H/L_h shall be greater than 0,2 (see Figure G.1 for definitions of H and L_h).

b) H shall be greater than 5 m in the case of Terrain Categories A and B, greater than 18 m for Terrain Category C.

The most accurate method of calculating topographical effects is by the use of dedicated software which is available to wind engineering specialists. This software utilises the theory first developed by Jackson and Hunt ("Turbulent wind flow over a low hill", Journal of the Royal Meteorological Society 101(1975, 929-955)), and since verified by a combination of modelling, wind tunnel, and field studies. The theory of Jackson and Hunt uses a linearized form of the boundary layer formulas in combination with Fourier series techniques to calculate the velocity perturbations induced by the underlying topography relative to an unperturbed reference velocity profile above flat terrain.

This approach is particularly recommended where overhead power lines are located in mountainous or topographically complex terrain, or where the wind speed up calculated by the manual method described above exceeds 30 %.

Where access to such software is not available, the following simplified approach may be adopted:

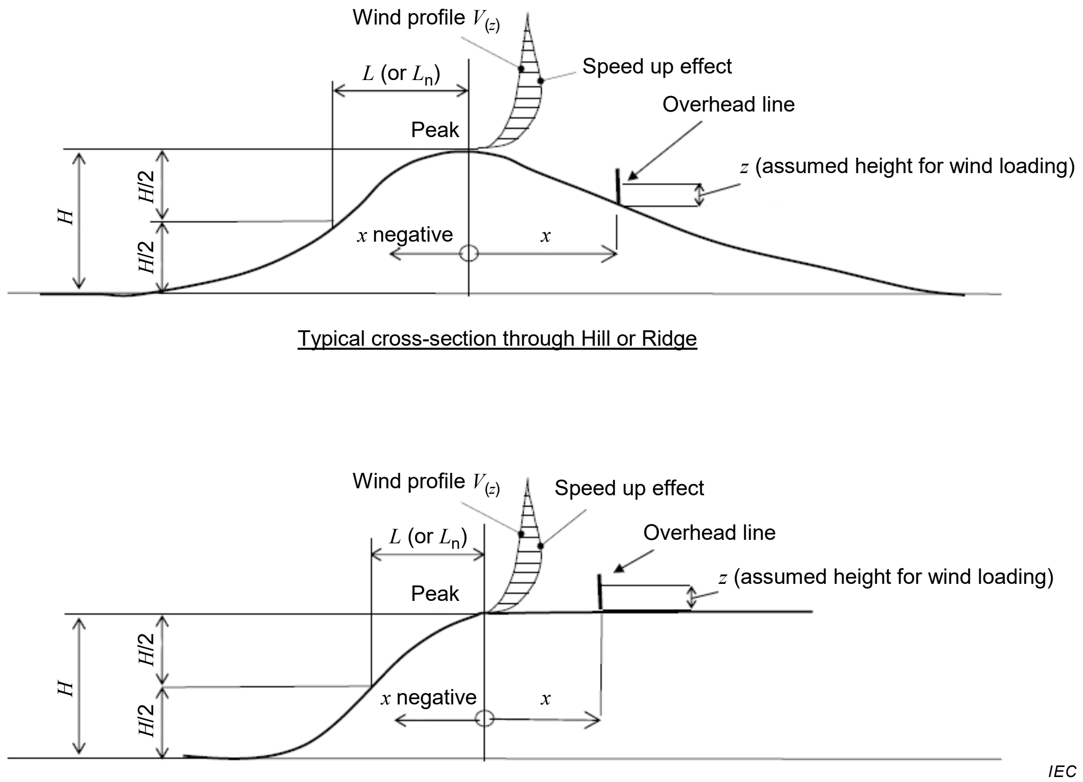

Figure G.1 – Diagram of typical topographical cross-section

S_{xz} is the wind speed-up factor at horizontal distance (x) from the peak and local height (z). (x is negative in the upwind direction, positive in the downwind direction.)

$$S_{xz} = (1 + S_1 \cdot S_2 \cdot S_3)$$

where

S_1 is determined from Table G.1 below;

S_2 = {1-abs(x)/($\mu \cdot L_h$)};

S_3 = exp($-\gamma z/L_h$).

For values of μ and γ, see Table G.1.

Table G. 1 – Values of μ and γ

Terrain type	$K_1/(H/L_h)$			γ	μ up-wind (x -ve)	μ down-wind (x +ve)
	Terrain category A	Terrain category B	Terrain category C			
Ridge or valley	1,55	1,45	1,3	3	1,5	1,5
Escarpment	0,95	0,85	0,75	2,5	1,5	4
Axisymetrical hill	1,15	1,05	0,95	4	1,5	1,5

G.2 Notes on application

For assessment of global wind speed up on a single support, reference wind speed V_r may be multiplied by effective speed up factor $S_{x,z}$, calculated using the above formulation, where x is the distance from the hill crest of the support, (positive in the downwind direction), and z is the height above ground of the centroid of the wind loading.

It is suggested that z can be taken as the centroid of the wind loading, which can generally be assumed to be average attachment height of the phase conductors, using parameter values representative of the whole wind span supported by the structure(s) under consideration. If there is substantial variation of $S_{x,z}$ over the wind span then values of $S_{x,z}$ may need to be calculated at a number of points and a mean value adopted. Wind speed up effects are generally strongly dependant on wind direction, and a number of different directions may require investigation.

As an alternative, as wind speed-up decreases rapidly with height above ground, it may in some cases be considered appropriate to consider this variation in design. In this alternative approach design wind pressures for all elements of the overhead line, including the different sections of the structure, may be multiplied by different values of $(S_{x,z})^2$ calculated using values of z representing the mean height of the various elements.

As this topic is continuously developing, it is possible that new methods may be available for studying wind turbulence, especially in the case of gust wind enhancements behind steep mountain terrain. Thus, future editions of this standard will be updated accordingly.

Bibliography

Cigre Technical Brochure 178: *Probabilistic Design of Overhead Transmission Lines*

Cigre Technical Brochure 273: *Overhead Conductor Safe Design Tension with Respect to Aeolian Vibrations.*

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