CHAPTER 3

PROBABILISTIC ESTIMATION OF TRANSMISSION TOWER COMPONENT STRENGTH

3.1 INTRODUCTION

The determination of transmission structure strength is a complex problem since reaching the ultimate load in an individual member may not necessarily constitute failure of the structure. However, certain members, such as the legs or body verticals in steel lattice towers are very critical to the integrity of the entire structure and attainment of the ultimate capacity for these critical members is indicative of the structure strength under loadings governing these members. It is of interest to evaluate the strengths of other less critical members of the structure to ascertain the overall consistency of the reliabilities for a variety of members in the structure. An investigation of the reliabilities for different members can be helpful for a number of reasons.

1. As a decision-making tool in identifying members that have unusually high safety indices and, therefore, are overconservative; or identifying critical members that have small safety indices and thus are detrimental to the overall safety or reliability of the structure.

2. To determine the minimum safety index for critical structure members that can be used as a measure of general structure reliability or in the development of a probability-based loading agenda for the design of future transmission lines.
For these reasons, it is necessary to establish the statistical parameters for component strength of various types of members and connections that make up a transmission structure.

The actual strength of a member is a random variable that can be described in terms of a probability density function. The distribution is governed by variations in material properties, fabrication and erection that naturally occur from member to member. The nominal strength of members, for various end conditions and slenderness ratios, is widely calculated using the design equations in the ASCE Manual-52 (1988) and IS:802 (1977, 1989). The mean, coefficient of variation and exclusion limit of the strength for various angle members used in transmission line structures summarized by Mozer et al. (1984) are shown in Table 3.1.

Table 3.1: Strength Parameters for Steel Members Typically used in Transmission Structures (Mozer et al. 1984)

<table>
<thead>
<tr>
<th>Member type</th>
<th>Mean Strength</th>
<th>Coefficient of Variation</th>
<th>Exclusion Limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-slender angle in compression</td>
<td>1.16 $R_n$</td>
<td>0.22</td>
<td>26</td>
</tr>
<tr>
<td>Slender angles in compression</td>
<td>1.21 $R_n$</td>
<td>0.18</td>
<td>17</td>
</tr>
<tr>
<td>Compression angles in X-bracing systems</td>
<td>1.25 $R_n$</td>
<td>0.14</td>
<td>8</td>
</tr>
<tr>
<td>Angles in tension</td>
<td>1.10 $R_n$</td>
<td>0.11</td>
<td>20</td>
</tr>
</tbody>
</table>

In Reliability-Based Design (RBD), the minimum description necessary for any kind of resistance includes the first two moments, i.e., the mean and coefficient of variation, are the only statistics computed for a given data set. The tacit assumption is often made that the data are normally distributed, although other distributions very often better fit resistance data. Once the moments are known any desired exclusion limit can be easily obtained if the distribution type is known.
3.2 VARIABILITIES AFFECTING COMPONENT STRENGTH

The design of a structure is usually performed by analyzing the structure as a complete entity followed by checking the strength of each component (member, connection) separately. The strength of a component may be defined for the entire member (e.g., buckling), for each cross section along the member’s length (e.g., flexure), for elements of a cross section (e.g., local buckling), or for connectors which, along with portions of the connected members, form the connections. Resistance checking is performed by defining the limit states which must be satisfied, determining the equations which relate material properties, geometry and applied load effects for each limit state, and then revising component size until each limit state is satisfied. Since changing member size changes member stiffness as well, reanalysis followed by rechecking is very often required for statically indeterminate structures.

The resistance of a component as computed by the designer is a function of:

a. Mechanical properties (Stiffness and strength) of its constituent materials
b. Geometry
c. Derived or empirical equation relating mechanical properties, geometry, and limit states considered.

The material properties are inherently variable, the cross section dimensions and other geometry can vary due to fabrication practices, and uncertainty exists in how well the equation used models the real behaviour of the component. Thus component variability is a complex function of all these variabilities plus possibly other variable factors.

The nominal resistance $R_n$ is the resistance computed using the nominal values of properties and geometry and the equation considered best able to predict
resistance. The real resistance, $R$, and the nominal resistance can be assumed to be related as:

$$ R = R_n (M* F* P) $$

(3.1)

Where,

$M = \text{A measure of the material variability obtained by testing components in which the materials are allowed to vary but the geometry is held as its nominal value, with M computed as the ratio of test to computed nominal value.}$

$F = \text{A measure of the variability due to fabrication practices obtained by testing with material properties at their means and geometry allowed to vary.}$

$P = \text{The 'professional' factor which is a measure of the inability of the design equation to properly predict real behaviour, obtained by testing sections will all properties and geometry at their nominal values.}$

Equation (3.1) is reasonable for the case where $M, F,$ and $P$ are independent random variables. The mean values of $M, F,$ and $P$ will differ from one when systematic errors exist. For example, the mean value of $M,$ $\bar{M},$ will exceed one when nominal material properties are based on a lower five percent exclusion limit.

As a first approximation, the mean value of resistance, $R$, is given by:

$$ R = R_n (\bar{M} \times \bar{F} \times \bar{P}) $$

(3.2)

The coefficient of variation of $R$ is given by

$$ V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} $$

(3.3)
3.2.1 Quantitative estimates of component variabilities

3.2.1.1 Material and geometric effects (M)

The variation in yield strength is taken as the main component of variability due to material and geometrical properties of angles. In Australia, Kitipornchai et al. (1984) mentioned the following value for $M$:

$$\tilde{M} = 1.14; \quad V_M = 0.06$$  \hspace{1cm} (3.4)

In the USA, a comprehensive study of the variability of steel as a material was made by Galambos et al. (1978) and mentioned the following values for $\tilde{M}$:

$$\tilde{M} = 1.10, \quad V_M = 0.11$$  \hspace{1cm} (3.5)

3.2.1.2 Fabrication effects (F)

There are experimental data on the effect of fabrication on the strength of lattice towers. The following recommendation was made by Ravindra et al. (1978a) on the fabrication effects.

$$\tilde{F} = 1.0; \quad V_F = 0.10$$  \hspace{1cm} (3.6)

3.2.1.3 Component strength analysis effect (P)

In practice, structural analysis of a lattice tower treats the tower as a space truss problem, and the stability analysis is only considered at the individual compression member level. The design capacity of each member is usually determined with the formulae of ASCE Manual-52 (1988) and IS:802 (1977, 1989). They have been used for many years in the design of transmission towers.

Recent full scale tests by the Electric Power Research Institute (EPRI) at the Transmission Line Mechanical Research Facility (TLMRF) clearly showed that the current design methods cannot be used to predict accurately the capacity of a transmission tower. In fact, it may be argued that tower testing is still a necessity
because analytical methods are not yet capable of predicting the capacity of a lattice tower within the desired accuracy (CIGRE, SC-22, 1990).

Full scale tests of TLMRF also showed that the current ASCE Manual-52 (1988) design formulae are not always appropriate and so, failures often involve a group of members rather than individual members.

Data for axially loaded angles were obtained by Strang et al. (1922), Kennedy et al. (1972) and Kitipornchai et al. (1984) and they concluded that the ASCE Manual-52 (1988) is unconservative by about 7% for slenderness ratio between 50 and 100 when compared with experimental data. In Australia, Pham et al. (1992) showed that both the current Australian Telecom (AS3995, 1991) and the new Australian Standard (1991) specified strength curves are unconservative by 12% and 30%, respectively, for the slenderness ratio range 100-150 and for double-bolted angles, the ASCE Manual-52 (1988) is generally less conservative than the Australian current Telecom procedure (AS3995, 1991).

Bathon et al. (1993) carried out a vast number of tests on equal, unequal, short and long angles for the investigation of angle capacity and showed that the actual angle capacity for the majority of specimens is smaller than the predicted angle capacity using the recommendation of the ASCE Manual-52 (1988). It was concluded that the design recommendations of the ASCE Manual-52 (1988) produce safe towers, but can overpredict individual member capacity.

### 3.2.1.4 Overall effects

In Australia, Pham et al. (1992) obtained the values of $\frac{R}{R_n}$ for leg member with a slenderness ratio in the range of 50-100 (i.e. non-slender angles) as:

a. For the current Telecom (1991) Procedure:
   \[ \frac{R}{R_n} = 1.81, \, V_R = 0.17 \]
b. For the AS 3995 (1991) procedure
\[ \frac{\bar{R}}{R_n} = 1.11, V_R = 0.16 \]

In the USA, Mozer et al. (1984) mentioned the following estimates for non-slender angles (i.e. L/r ratio range from 50 to 100) in compression with an exclusion limit of 26%.
\[ \frac{\bar{R}}{R_n} = 1.16, V_R = 0.22 \]

### 3.3 VARIATION OF NOMINAL STRENGTH FOR DIFFERENT EXCLUSION LIMIT

The exclusion limits of the currently used strengths, \( R_n \) vary widely. Figure 3.1 shows the variation of ratio of nominal strength to mean strength for different coefficient of variation of strength and exclusion limits. It can be noted that with the increase in exclusion limit, there is an increase of nominal strength and also nominal strength decreases with the increase of coefficient of variation of strength irrespective of the exclusion limit. It was found that the exclusion limit for the strength of a steel lattice tower member based on the strength formulae in the ASCE Manual-52 (1988) and on real test is in the range of 5-25%. The current strength calculation procedures is not an obstacle to the production of reliability consistent designs with the LRFD method. The ASCE Manual-74 (1991), defines a value of 5% for the exclusion limit for tower components with a coefficient of variation of strength of 15% for a reliable component design. On the other hand, the IEC-826 (1991) document specifies a value of 10% for the exclusion limit.

### 3.4 POST-BUCKLING FAILURE STRENGTH OF TOWER MEMBERS

Compression member performance can be classified into three phases. They are,

a. Elastic
b. Inelastic
c. Post-buckling
FIGURE 3.1 DESIGN GRAPH FOR THE RATIO OF NOMINAL TO MEAN STRENGTH UNDER DIFFERENT EXCLUSION LIMITS

FIGURE 3.2 TYPICAL LOAD-DEFORMATION BEHAVIOUR OF A TOWER MEMBER UNDER COMPRESSION
The post-buckling region begins when there is no increase in load resistance for increased axial shortening of the member. In the probabilistic context, a member can fail in tension or compression and their strengths are treated as random variables. When members fail in compression the compressive strength of the members drops to a fraction \( \eta \) of the original strength and this fraction \( \eta \) is called as the post-buckling factor is also a random variable. The magnitude of the post-failure factor \( \eta \) depends on the slenderness ratio \( (L/r) \) of the member. A typical post-buckling behaviour of a compression member is shown in Figure 3.2. The post-buckling factor \( \eta \) can be a normally distributed random variable or lognormally distributed variable. In addition to random variation, the post-buckling strength factor \( \eta \) can be assumed as a deterministic coefficient of 1.0 i.e. ductile failure, a deterministic coefficient of 0.0, i.e. brittle failure. According to Smith (1984) if the post-buckling strength factor is treated as a random lognormally distributed variable, the mean value of 0.4 and coefficient of variation of 10% is the most realistic one. The details of the post-buckling formulation of compression member failure will be discussed in the subsequent chapters.

### 3.5 DESIGN STRENGTH FORMATS

The variability of steel strength can be defined at both the material and the component levels. A knowledge of the variability of the material properties of steel is fundamental input to RBD. Variability of component behaviour can be considered during the development of the design equations.

**ASCE Format**

The ASCE Manual-74 (1991) recommends an exclusion limit of 5% to calculate the nominal component strength with a coefficient of variation of strength of 15%. It argues that this limit is low enough to be chosen as the value at which the component can be proof loaded.
IEC Format

The International Electrotechnical Commission (IEC/TC 11,1991) document specifies that for reliability analysis, the load compares to the 10% exclusion limit of strength. Most National Codes of Practice, manufacturers catalogues, will quote a value which will be accepted as a characteristic strength. The characteristic strength with a factor can be equated to a strength with a 10% exclusion limit and a coefficient of variation of strength of 5-10%.

A comment on both the formats

If we consider a coefficient of variation of strength as 20%, the difference of strength prediction between above two formats is only about 9%. The conclusion is that the two methods will give similar tower design. However, the ASCE format is preferred since it is an accepted format by the structural design community and on the other hand, the IEC format is yet to be finalized. In India, the variability of the steel is not yet established especially for tower materials. There are large number of tower testing stations available in the country. Prototype testing should be performed to confirm whether strength formulae for the tower members satisfy at 10% exclusion limit or 5% exclusion limit, as far as lattice towers are concerned.

On the other hand, in order to derive full benefit from past experience, a detailed analysis of the strength of existing lines should be done in order to formulate new proposals. Most of the existing lines can withstand atleast 2 to 3 times climatic loads, specified by the standards, without damage because of built-in overload factor and safety factors. Consequently, it is very important to assess the limit strength of existing lines and towers in order to formulate new design codes.