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Recent Failure of a Transmission Line Due to Ice in Newfoundland

Asim Haldar
Newfoundland and Labrador Hydro
P.O. Box 12400
St. John's, Newfoundland, Canada
A1B 4K7

ABSTRACT

The present study deals with the recent failure of an existing steel transmission line which runs on the west coast of Newfoundland. The line was built in 1966 and commissioned in 1967. Since its commissioning, the line has experienced severe ice loadings almost every year including a major failure in 1987. The line crosses a relatively high elevation (1800-2000 feet plateau) known as Hinds Plain and is largely exposed to a northerly storm wind direction. Recent Meteorological Studies have been conducted in the general area of Hinds Plain, and indicate that the ice accretion potential far exceeds the design loadings and even a five (5) to ten (10) year return period ice load exceeds the line's ultimate failure capacity. In order to estimate the revised ice load, a time dependent numerical in-cloud ice accretion model has been used. Model input parameters include wind speed, air temperature, liquid water content, median droplet diameter, and conductor diameter. The model accomodates variable ice density and dry and wet growth conditions. To upgrade this line with regard to the commonly acceptable level of fifty (50) year return period loading criteria, a cost-risk analysis is carried out in order to determine the optimum design span. Result of the study indicates that the shortening of the existing span by adding structures is the most economical solution as compared to the re-route alternative.

INTRODUCTION

The 230 kV transmission line, which runs from Buchans to Massey Drive (Fig. 1) was built in 1966 and commissioned in 1967. Since its commissioning, the line has experienced severe ice loadings almost every year. The line crosses a relatively high elevation (1800-2000 foot) and open area, known as Hinds Plain. The line is 84 km long and crosses the Grand Lake on the west side of Newfoundland. This crossing starts at approximately 57 km from Buchans. In this paper, the line east of Grand Lake is defined as the Central section whereas the line West of Grand Lake is defined as the Western section. The line consists of two hundred and sixteen (216) steel towers with an average span of 1250 feet (380 m). Most of these towers are suspension type guyed-v structure. Several large ice accumulations have been actually observed and there were two major line failures in 1980 and 1987. Failure prior to 1987 has been well documented in Ref.1. However, a summary is provided below:

1. 1967 - conductor came to within 1.2 m of ground line.
2. 1980 - bridge members of one structure in the Central section failed resulting in a ten (10) day outage and a repair cost of $40,000; conductor was damaged at several places in the Central section.
3. 1981 - conductor came to ground line.
4. 1987 - seven structures failed in a cascade situation in the Central section with broken conductor resulting in an outage of 63 days at a cost of $500,000 dollars, approximately.

As a result of two (2) major line failures within eight (8) years in the Central section and conductor touching the ground in the Western section, Transmission Design Department of Hydro undertook a detailed study on the assessment of the existing line reliability and the courses of action that are necessary to increase the level of reliability of this line.

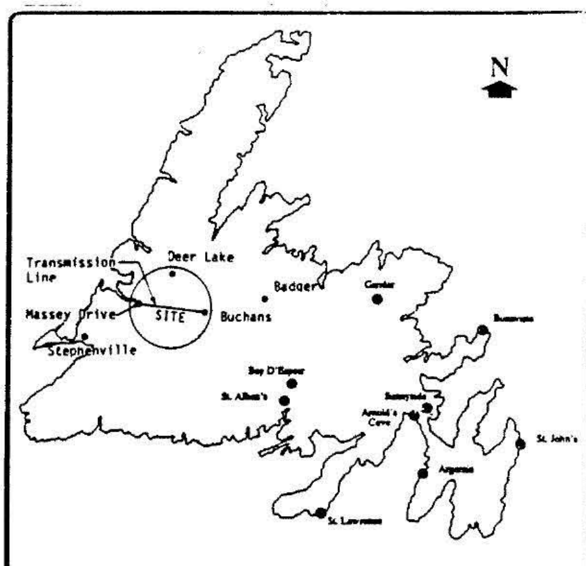

FIGURE 1.- SITE LOCATION PLAN

Two basic scenarios are considered in this study to improve the reliability to the commonly acceptable level of fifty (50) year return period loading criteria. These scenarios are: addition of mid-span structures and rerouting of the line at selected locations, in both sections.

In general, part of the Central section (approximately 23 km) passes through higher elevations which will have lower temperatures and longer icing seasons than lower elevations (eg. between Buchans Terminal Station and first 19 km of the line). The line in the Central section has an average conductor elevation of 1800 feet (550 m) and largely exposed to stronger winds. It is well known that higher elevation coupled with strong wind at exposed sites often produces severe cloud icing (rime and in-cloud glaze) during a storm. This also means higher liquid water content, longer storm duration and perhaps also larger droplet sizes. All of these factors combine to produce high ice loading. On the other hand, topographic shelter at or the exposure of the site in relation to the storm wind direction has the effect of reducing both ice loads and wind speeds. Possible relocations of the route in both sections were also considered in this study. Average elevation of the proposed reroute in the Central section, is 1640 feet (500 m), while in the Western section, it is 1300 feet (400 m). Although there are reductions in average elevations in both sections, it appears that the reduction in the Central section is only marginal ten-percent (10%). The exposure of the Central section reroute option is still severe in relation to the storm wind direction (northerly), and reduction in the wind speed will be less than one-percent (1%). Based on a site visit it was decided that relocation of the route in the Central section will not reduce ice accretion in any significant way. However, reduction in the average elevation in the Western section due to the proposed rerouting is twenty-five percent (25%) and corresponding reduction in the wind speed is fifteen percent (15%). Also, the proposed reroute is relatively sheltered, and therefore, will reduce the ice accretion to a significant degree.

ANALYSIS OF DECEMBER 1987 STORM (Ref. 2)

On December 30, 1987 seven (7) structures failed in a cascade situation as a result of severe ice storm. This section describes the analysis of this storm with an ice-accretion model.

The Makkonen model (Ref. 3) for ice accretion on conductors was used to simulate the potential accumulation of rime ice from in-cloud conditions between 000 GMT on December 25, and 1500 GMT on December 30, 1987. The model was applied to three (3) data sets - Deer Lake, Badger, and the extrapolated upper air data from Stephenville. In Ref. 2, it was shown that all the reported weather conditions indicated that the ice accumulation was from in-cloud rime icing. Therefore, the model was used to simulate only in-cloud icing conditions for the duration of the event.

Fig. 2 presents a time series plot of wind speed and air temperature values for December 25 to 31, 1987. Two upper air data sets were available per day, and therefore, the derived values were considered to be constant over each 12 hour period between upper air measurements.

As no information on the liquid water content of the air and median volume droplet diameter of the water droplets in-cloud was available, constant values of 0.3 g/m^3 and 20 microns, respectively, were used for each model run. These represent relatively high values for each parameter, based on the limited measurements and laboratory investigations carried out to date for in-cloud icing conditions. To determine the degree of variation in the model associated with these two parameters, the model was rerun for the extrapolated upper air data using different constant values. For liquid water content, values of 0.2, 0.3, 0.4, and 0.5 g/m^3 were tested, while maintaining the median droplet diameter at 20 microns. For the median volume droplet diameter, values of 10, 20, and 30 microns were tested, with a constant liquid water content of 0.3 g/m^3.

Upper air data were first extrapolated from the Stephenville weather station to the line site using the gradient wind model, and subsequently, entered as one of the input parameters for the ice accretion model (Ref. 3). Hourly wind speeds from Deer Lake and Badger were extrapolated to the line site (Refer Fig. 1) using a gradient height and terrain roughness parameter of 335 m and 1/5.5, respectively, for each of the stations and a gradient height and terrain roughness parameter of 250 m and 1/10, respectively, for the line site. An adiabatic lapse rate of -0.65°C per 100 m increase in elevation was applied to extrapolate station air temperatures from the station elevation to a conductor elevation of 2000 feet (610 m).

Based on the sensitivity study, Fig. 3 depicts the variation in predicted ice thickness with regard to the two parameters such as liquid water content and median volume droplet diameter. It is noted from Fig. 3 that the choice of droplet diameter has a significant effect on the predicted ice thickness. However, the model results are less sensitive to the variations of liquid water content. Considering the severe exposure and the elevation of this specific site, a value of twenty-five (25) microns is assumed for the droplet diameter size to quantify the ice thickness.

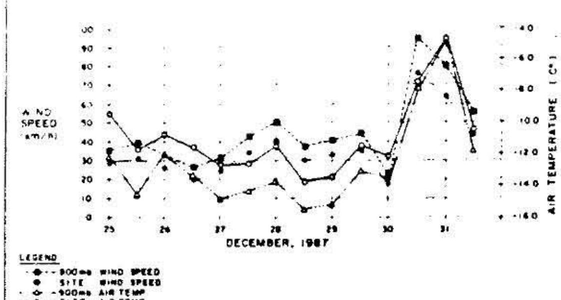

FIGURE 2
AIR TEMPERATURE AND WIND SPEED FOR THE 900mb LEVEL AND SITE, DEC. 25 TO 31, 1987

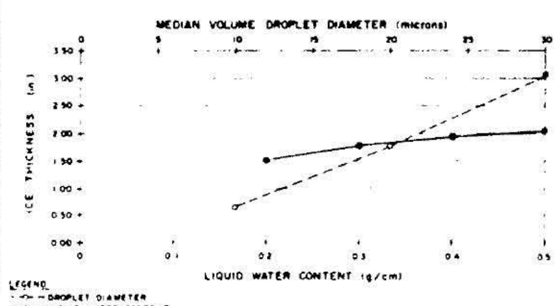

FIGURE 3
VARIATIONS IN PROJECTED ICE THICKNESS WITH DROPLET AND LIQUID WATER CONTENT

CLIMATIC LOADING

Original Design Criteria

Prior to the Bay d'Espoir Power Development, the basic criteria for the design of transmission lines built in Newfoundland was the CSA (Canadian Standard Association) "Heavy" loading of 0.5 inch (13 mm) radial ice combined with a 73 mph (117 kmh) gust wind. This criteria was moderately successful, although failures had occurred, particularly on the eastern side of the Island. For the distribution and communication circuits, the usual means of rectifi- cation of failures due to inclement weather was to rebuild with shorter spans. Table 1 presents the original design loadings used for high voltage transmission lines on the Island during the Bay D'Espoir Power development.

TABLE 1
Design Wind and Ice Loads for
Bay d'Espoir Power Development

Load Zone	Radial Ice inch (mm)	Gust Wind Speed mph (km/hr)	Temp °F (°C)	Max. Cond. Tension % UTS
Nominal Ice	1.0 (25)	0 (0)	0.0 (-18)	70
	0.5 (13)	73 (117)	0.0 (-18)	50
	0 (0)	110 (176)	0.0 (-18)	50
Heavy Ice	1.5 (38)	0 (0)	0.0 (-18)	70
	1.0 (25)	73 (117)	0.0 (-18)	50
	0 (0)	110 (176)	0.0 (-18)	50

Following Table 1, this line was originally designed with "Heavy Ice" for the Central section and the "Nominal Ice" for the Western section. In order to derive the revised climatological loading on the line, basic data were taken from the MRI report (Ref. 4) prepared for the HVDC transmission line route. MRI report uses the climatological data from the Buchans station operated between 1953-1964.

Extreme Wind

The 5-year, 10-year, 25-year and 50-year return period wind speed values for Buchans are shown in Table 2. These wind speed values from Buchans were taken to determine the extreme wind speeds on the Hinds plain. A power law (Ref. 5) formula was used to extrapolate the extreme wind speed values on the Plain and is shown below:

$$V_{ZH} = V_{ZB} \, (Z_H/Z_B)^{T} \qquad \ldots (1)$$

where V_{ZH} = extreme wind speed on the Hinds Plain

V_{ZB} = extreme wind speed in Buchans

Z_H = elevation over Hinds Plain (typical value 2000 feet)

Z_B = elevation at Buchans (946 feet)

T = terrain roughness factor (typical value 0.10 for open area)

TABLE 2 Return Period Values of Maximum Gust (Ref. 6) and Sustained Wind Speeds (mph) (Extrapolated from Buchans to Hinds Plain)

	Sustained Wind Speed		Gust Wind Speed
Return Period In Years	Buchans (Elev. - 946 feet)	Hinds Plain (Elev. - 2000 feet)	Hinds Plain (Elev - 2000 feet)
5	58	63	88
10	62	67	93
25	67	72	100
50	70	75	104

Extreme Ice

Original ice data were taken from MRI report (Ref. 4) for Buchans (1953-1964) and subsequently adjusted for the field observations, model predictions and estimated ice thicknesses based on the line failures in 1967, 1980 and 1987, respectively. Extreme value analysis were carried out for both sets of data and results are shown in Fig. 4. A 50-year ice thickness is estimated as 5.70 inches (145 mm) rime (specific gravity = 0.50), which is equivalent to a glaze ice thickness of 3.00 inches (75 mm) with a specific gravity of 0.90. Based on the above and the observed ice thickness values (in-cloud glaze) of 44 mm to 63 mm, it is recommended that a 50-year ice thickness of 3.0 inches (75 mm) glaze ice be used for the Central section. For the Western section, similar ice thickness was used both for the existing and the reroute sections.

Combined Wind and Ice

Estimate of the ice load based on the failures of conductor and tower indicate that 1.75 inches (44 mm) to 2.0 inches (50 mm) ice will be a very frequent loading on this line (1 in every 5 to 8 years). Maximum wind will most likely occur during the winter months on the Hinds Plain. Determination of the combined wind and ice is based on the joint probability of occurrence of wind and ice which will produce a 50-yr. return period equivalent load. Based on the analysis of 1987 storm, several freak loads were used to simulate the conductor breakage condition and it was estimated that the existing conductor will break somewhere between 2.0 inch (50 mm) to 2.25 inch (56 mm) ice. It is determined 2 inch ice (3.6" equivalent rime) is occurring every 9 years from Fig. 4. Therefore, corresponding return period of combined wind speed to produce a 50-year load is (.02/.11 = 0.18 or 1 in every 5.5 years).

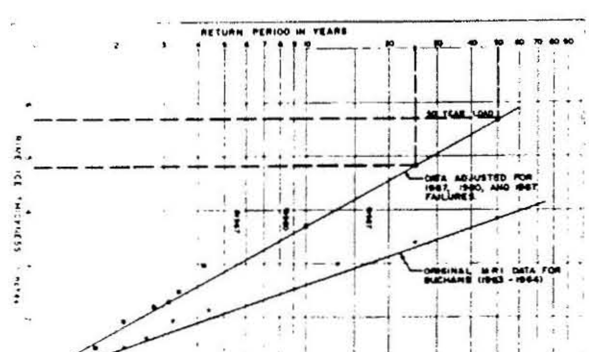

FIGURE 4
EXTREME VALUE ANALYSIS OF RIME ICE DATA
(DATA TO BE ADJUSTED FOR 1967, 1980 & 1987 STORMS)

RELIABILITY ANALYSIS

In the reliability analysis, interference between the strength and stress (effects of various loads) is taken into account using the mathematical theory of probability. Failure probability is determined in terms of a dimensionless quantity often termed as reliability index (β). Normally, probability distribution functions are assumed for resistance (R) and load effects, (Q). The computed, β, value is very sensitive to the shape of the distributions in the upper tail range. The conversion of loads to load effects is carried out through suitable structural analysis (Ref. 7). A simplified relationship is established between the failure probability of the line to the annual probability of failure of the structure assuming that each structure in the line is utilized one-hundred percent and each load event covers the entire transmission line (Ref. 8).

In order to determine the relative reserve strength of a typical failed tower and the conductor in the Central section, computation was carried out based on the corresponding ice-weight span of the structure and the ruling span for the conductor. Details of the computation are given in Ref. 8. Results from the computation indicate that the existing conductor will be the first to fail under extreme ice before any member in the bridge fails. This points out that to improve the reliability of this line, one must ensure that the conductor is stronger than the structure under extreme ice loading situation. The preferred sequence of failure is to loose a member in the bridge rather than the conductor. The above calculation is based on the factor of safety (deterministic) approach and also assumes that the conductor fails at its 100% UTS (Ultimate Tensile Strength). However, experience shows that failure will normally occur well below its full rated strength because of the bending and compression effects of the clamp and this may be somewhere between 90% to 95% UTS. Field reports indicate that the conductor broke 100 feet from the clamp of the failed tower. This tower has a factor of safety of 1.20 compared to the conductor factor of safety of 1.0 based on 2.0 inch (50 mm) ice load. The typical value of the limit vertical load for a tangent tower has been taken as 14,000 lb (62 KN) per phase based on the original design criteria.

Evaluation of Strength

In the present analysis, force in a typical member has been treated as a deterministic quantity; however, basic parameters for determining the force are based on an extreme value analysis such as the ice thickness with a specific return period value. It is also assumed that the strength of a member is a random variable and related to the nominal strength (R_n) as:

$$R = MFPR_n \qquad \ldots (2)$$

where M = variability due to material
F = variability due to erection and fabrication
P = professional factor that represents the uncertainties in the strength theory.

and R_n = nominal strength based on Ref. 9

IF M, F, and P are uncorrelated random variables, the mean value and coefficient of variation for the strength can be approximated as:

$$\bar{R} = \bar{M}\,\bar{F}\,\bar{P}\,R_n \text{ and } V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \qquad \ldots (3)$$

where V_R = coefficient of variation of strength.

Design Load Cases

The original towers were designed for five (5) basic load cases. These were ice only, transverse wind only, transverse combined wind and ice, longitudinal ice unbalance and stringing/maintenance load. However, during the course of study it appeared that the original tower loadings did not reflect fully the actual loading conditions that have occurred in Hydro's system during the last twenty (20) years. One of the loading conditions, unbalanced vertical ice loads, which was originally not considered in the design, was found to cause failure, particularly in the central section of the bridge sections of the tangent tower. In view of this, an upgraded tower type similar to the existing guyed tangent tower was also considered in the analysis. The tower was upgraded by modifying some of the selected members in the bridge to carry the unbalanced vertical loads. Increase in the tower weight was forty (40) kg which is only 2% of the total weight. In the present study, both existing and upgraded tangent towers were analyzed under various basic climatic loading conditions and load combinations derived from these basic loads. The load cases such as diagonal and longitudinal winds were not considered here because they will not govern the design situation. Stringing and maintenance loads were not considered in the present analysis although these load cases can be included in the design process.

Structural Analysis

Based on the stiffness analysis, structural responses such as joint deflections and member loads are computed under various load combinations. Guy tensions are determined based on a non-linear analysis using iterative techniques. Fig. 5 depicts the computer model of a typical guyed-v tower with 321 nodes and 961 members.

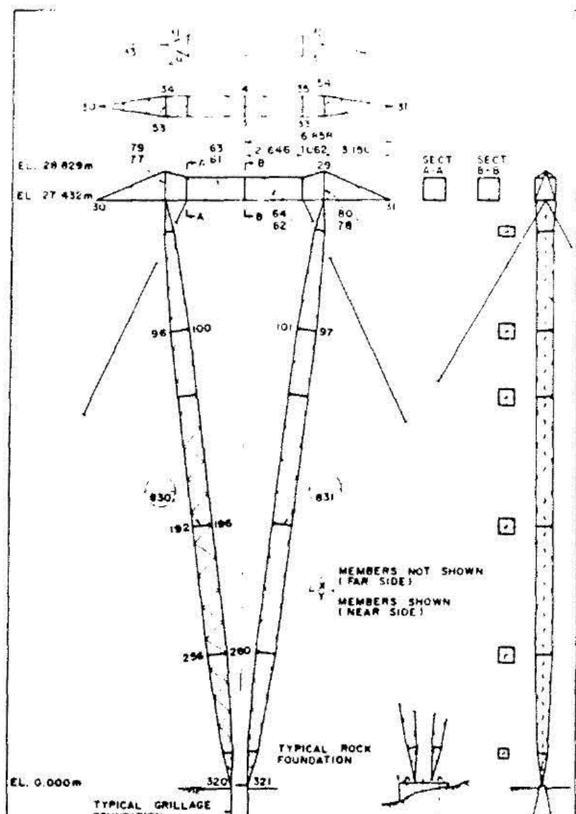

FIGURE 5
COMPUTER MODEL FOR A GUYED TANGENT TOWER

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System Reliability of a Guyed-V Tower

Probability of failure of the tower has been computed based on the assumption that an individual member fails either in tension or in compression mode. Therefore, these two events are mutually exclusive under a particular load case. Members have been grouped according to the particular mode of failure and total reliability of the tower is given as:

$$R_{Tower} = \left(\prod_{i=1}^{n} R_{ci}\right)\left(\prod_{j=n+1}^{m} R_{Tj}\right) \qquad \ldots (4)$$

in which,

R_{ci} = reliability of an individual member, i, in compression

R_{Tj} = reliability of an individual member, j, in tension

m = total number of members under consideration

and R_{Tower} = reliability of the tower

Symbol, Π, represents the product relationship.

Assessment of Lifetime Probability of Failure

A comprehensive estimate of the tower failure probability under all possible load cases should include the probability information related to the occurrences of these load cases. Assuming the load case events are independent, the estimate of the tower lifetime failure probability is given by:

$$P_{f,T} = \sum_{i=1}^{M} P(N_i)\, P_{f,i} \qquad \ldots (5)$$

where $P_{f,i}$ is the probability of failure given that the "load case, i" occurs, $P(N_i)$ is the probability of occurrence of this load case and 'M' is the number of load cases. In this study, a total of nine (9) load cases are considered. Seven (7) of these load cases are related to ice-only loadings while the remaining two load cases are extreme wind and combined wind and ice loads, respectively. Table 3 presents these load cases.

For example, under ice loading, a suspension tower may be subjected to either a vertical unbalance (bending mode of failure) or a longitudinal unbalance (torsional mode of failure) and exact determination of the probability of occurrences of these loads or load combinations is extremely difficult. Some subjective preassigned probability of occurrence values, $P(N_i)$, need to be used with judgement. Field experience shows that typical ice residence time is two (2) to three (3) weeks and probability of complete ice shedding within this residence time on one phase, as opposed to other two phases will have an extreme low value because of the open nature of the terrain and higher elevation. One would expect here a gradual ice shedding and therefore a low probability value of $P(N_i)$, should be used when computing the lifetime probability of failure of the tower under all possible combinations of ice loadings. Similar arguments can be made with longitudinal unbalance loadings because of the reduction, and more uniformity in the adjacent span lengths, based under new design criteria. Subjective probability values for different ice loading combinations are used in Table 3 to compute the lifetime probability of failure of the tower for 800 feet (244 metres) span. Table 3 shows that the upgraded tower has greater 'β' value compared to the existing one indicating an increase in reliability level. This is expected because of the increased member sizes in the bridge level and associated higher strength steel.

Total lifetime failure probability of the tower under all primary load cases is computed as follows:

$$(P_{f,T})_{Total} = \sum_{i=1}^{7} P(N_i)\, P_{f,i} + P_{f,8} + P_{f,9} \qquad \ldots (6)$$

where $P_{f,8}$ and $P_{f,9}$ are defined as the probability of failure of the tower under extreme wind and combined wind and ice loads, respectively. Values for $P_{f,8}$ and $P_{f,9}$ have been obtained for both types of towers (Ref. 8) and are shown in Table 4. Table 4 presents the overall lifetime and annual failure probabilities for 800 feet (244 metres) span. Following Ref. 10, the target reliability (or the annual probability of failure, P_a) is set as 1/100 for a 50-year return period of loading (reliability level-one). Table 4 shows that existing tower with a 800 feet (244 metres) span under the proposed 50-year extreme loading will meet the reliability level-one criteria (annual P_a = .0096). However, with the upgraded tower, this reliability level will be further increased as noted from Table. 4. Increase in the reliability (or decrease in the probability of failure) can be obtained further, when one reduces the design span to 700 feet (213 metres) level for both tower types (Ref. 8). Final determination of these design spans will be made based on a cost-benefit analysis as described in the following section.

TABLE 3 - Comparison of Lifetime Probability of Failure Under Ice-Only Loadings:

Design Ice Thickness (glaze) = 3.0 inch (75 mm)
Design Weight Span = 800 feet (244 metres)
Height of Tower = 90 feet (27.44 metres)
(Guyed-V shape)

Load Case	Description	$P_i = P(N_i)$	Existing Tower $P_{f,i}$	Existing Tower $(P_{f,T})_i$	Upgraded Tower $P_{f,i}$	Upgraded Tower $(P_{f,T})_i$
#1	Full Ice Load on Phases	0.80	0.385	.308	0.303	.2424
#2	Full Ice Load on two Phases, & Partial Load on one Phase, a = 0.60	0.08	0.337	.027	0.323	.026
#3	Full Ice Load on two Phases, & Partial Load on one Phase, a = 0.40	0.02	0.694	.0139	.0435	.00087
#4	Full Ice Load on two Phases, one Phase Bare, a = 0.0	0.02	0.99	.0198	0.285	.0057
#5	Full Ice Load on two Outside Phases, Middle Phase Bare	0.02	.0553	.0011	.025	.0005
#6	Full Ice with Longitudinal Ice Unbalances on all Phases	0.03	0.728	.02189	0.353	.0106
#7	Full Ice with Longitudinal Ice Unbalances on any two Phases	0.03	.733	.022	.606	.0181
		$P_{f,T}$ =		.4136		.3042
		β =		0.22		0.52

$P_{f,T}$ = Lifetime Probability of Tower
$P(N_i)$ = Probability of Occurrence of a Load Case, i
$P_{f,i}$ = Probability of Failure under Load Case, i
β = Reliability Index

TABLE 4 - Comparison of Lifetime Probability of Failure for Existing and Upgraded Towers Using All Load Cases

Design Span Length	Failure Probability: Existing Tower	Failure Probability: Upgraded Tower
800 foot (244 metres)		
	0.4136 (Table 3 - Ice Only Loadings)	.3042 (Table 3 - Ice Only Loadings)
	0.00444 (Load Case #8)	.0043 (Load Case #8)
	0.0617 (Load Case #9)	.0571 (Load Case #9)
Total $(P_{f,T}) = \sum_{i=1}^{9} (P_{f,T})_i$ (Eqn. 6)	0.4797	.3656
Reliability Index (β)	(0.05)	(.35)
Probability of Failure (P_a) - Annual	.0096 (1 out of 100)	.0073 (7 out of 1000)

ECONOMIC ANALYSIS (Ref. 11)

To determine the design span based on the optimum structural failure probability, economic analysis is carried out in which the initial cost of the line is balanced against the future failure costs.

The initial cost of the line is a function of the probability of failure of the line and decreases as the failure probability increases. Probability of failure of the line is assumed to be governed entirely by the annual failure probability of an individual structure and therefore, initial line cost is estimated based on the direct material and erection costs of the total number of structures for a given span and line length.

On the contrary, the cost of failure, which includes the restoration cost plus any damage costs, increases as the failure probability increases. Hence, there exists an optimum failure probability at which the total cost is minimum. Assuming that the failure occurs at time $t<T$, where T is the economic (design) life of the structure and denoting the present value of the future losses at H(t), one obtains the total cost as the sum of initial cost plus any future failure costs, as

$$C_T = C_I + (P_a \times PvF \times C_F) \qquad \ldots (7)$$

where C_F is the damage cost, P_a is the annual probability of failure and is related to the lifetime probability of failure, $P_{f,T}$ as shown in Table 4 and PvF is defined as the present value factor which is a function of net discount rate, i, (difference between the interest and inflation rates), economic life, T, and the annual probability of failure, Pa. Typical value of C_F has been reported by the Operations Division for the failure of this line in 1987 and this is estimated to be $0.50 million dollars, approximately (in 1987 dollars) for the Central section alone.

Choice of Optimum Design Span:

Table 4 presents the risk factors for the existing and upgraded towers with 800 feet (244 metres) design span. It is also noted that 800 feet (244 metres) design span will meet the minimum criteria of target reliability as described in Ref. 10. Ref. 8 presents the direct costs for addition of midspan structures for 700 foot (213 m) and 800 foot (244 m) spans and these are $7.9 and $7.3 million dollars, approximately, for the above two design spans. It is also assumed that the present value of the cost of failure is $1.0 million (based on recent statistics on costs of failure) for both sections. Therefore, total cost as described by Eqn. 7 becomes $7.92 and $7.39 million dollars, approximately. ($C_T = C_I + P_a \times PvF \times C_F = 7.3 + .0096 \times 9.086 \times 1.0 =$ $7.39 million, for an 800 foot span). This indicates that the existing tower with an 800 foot span (244 m) will meet the target reliability level of 1/100 as specified in Ref. 10. Considering the reroute option, total cost is estimated as $9.2 million dollars, approximately (Ref. 8). Therefore, shortening of the existing span with the addition of more structures provides an optimum cost ($7.39 million dollars). All cost optimizations were carried out based on the minimum number of surplus towers that will be available after the completion of the work.

CONCLUSIONS

The present study shows that the existing line, at present, has a high risk of failure. This is primarily because of the under estimation of the extreme ice loading over the Hinds Plain, in the original design. Analysis indicated that the existing conductor is not strong enough to withstand even the 50 mm (2.0 inch) glaze ice load which will probably occur every 5 to 8 years based on the updated prediction of the ice thicknesses. Reliability analysis indicates that the towers have extra reserve strength when compared to the conductor. This is contrary to the current design practice with regard to the reliability based design approach where the line is treated as a whole system, and tower, foundation, and conductor being its sub-system. System approach in conjunction with the cost benefit analysis indicates that the shortening of existing span by adding structures is the most economical solution for upgrading this line.

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REFERENCES

1. Assessment of the Design Parameters for TL 228 from Buchans to Grand Lake Crossings; Hydro's Report #3-2-23, 1983.

2. Probabilistic Ice and Wind Loadings for TL 228 (230 kV Transmission Line from Buchans to Massey Drive). Report #4-7-73 prepared by Compusult Limited for Transmission Line Design Department, Newfoundland and Labrador Hydro, 1988.

3. Makkonen, L. 1984. Modelling of Ice Accretion on Wires, Journal of Climate and Applied Meteorology, 23 (6); 929-939.

4. Meteorological Study of the Gull Island Transmission Line Route - Report prepared by Meteorological Research Inc., MRI 73FR 1131 California, 1973.

5. Davenport, A.G. 1979. Gust Response Factors for Transmission Line Loading. Proc. Fifth International Conference on Wind Engineering, Fort Collins, Colorado, pp. 899-909.

6. Sissenwine, N., Tattelman, P., Gronthman, D. and Gringorten, I. 1973. Extreme Wind Speeds, Gustiness, and Variation with Height for MIL-STD-210B, Air Force Cambridge Research Laboratory, Tech. Report 73-0560.

7. 'TOWER" Program - Structural Analysis of Latticed Transmission Tower (Self-Supported and Guyed Towers, developed in Hydro's operating system) by Transmission Line Design Group, Newfoundland and Labrador Hydro, 1987.

8. Haldar, Asim 1989. Probabilistic Assessment of the Design and Upgrading Work for TL 228 from Buchans to Massey Drive. Report #3-2-51, Newfoundland and Labrador Hydro.

9. Guide for Transmission Line Design. Published by American Society of Civil Engineers, NY., 1971.

10. IEC Document, Loading and Strength of Overhead-Lines. Prepared by IEC/TCII Committee, 1989.

11. Haldar, A.K. 1988. Reliability Based Design of Transmission Lines, Proc. Second International Symposium on Probabilistic Methods Applied to Electric Power Systems, pp. 13; San Francisco, EPRI EL-6555.