METHODOLOGY FOR CONDUCTING EXTREME EVENT MODEL SIMULATIONS

Prepared by

G&E Engineering Systems Inc.
California

April 2022 (Revision 1)

Principal Investigator
John Eidinger, P.E., S.E.

ABSTRACT

This report provides a methodology to examine the reliability of high voltage transmission lines due to extreme wind and ice loads. The inventory, hazards, fragility and models are organized to allow the end-user to do rapid (Level 1) and modestly detailed (Level 2) reliability analyses. The models allow the user to examine the reliability of existing transmission circuits that have been damaged in actual storms, to establish the probable causes and possible mitigation plans. The models provide default parameters, but allow the user to select customized parameters, such as tower over-strength, local geological conditions, variability in tower capacity, ranges of uncertainties in wind (ice) loads, either for individual towers or all towers.

This report also provides a benefit cost model to examine the cost effectiveness of upgrading actual transmission lines. The benefit cost model can be used to perform "what-if" analyses to consider the change in reliability should alternate design decisions be made, such as including closer spacing of dead-end towers, increasing (or decreasing) the design wind speeds and ice accumulations; as well as whether the incremental cost to build more-robust towers is cost-effective.

Appendices A and B provide reference maps showing wind speeds and ice thicknesses. These maps cover all areas in Canada and the United States. Appendices C and D provide common hurricane and tornados scales. Appendix E provides "expert-based" fragility curves for transmission towers and related facilities due to high winds.

Databases are discussed that show past ice storms, tornadoes, wind storms, hail storms and past earthquakes.

Notice: the electronic files (spreadsheets and database models) described in this report were developed by G&E Engineering Systems Inc. These electronic files are not included with this report. Contact G&E if you wish to obtain copies of the electronic files. All electronic files require entering into a suitable agreement with G&E, and require third party software developed by Microsoft and Claris.

Keywords:
Reliability, transmission towers, wind, ice, benefit cost analysis.

ACKNOWLEDGEMENTS

The following electric utilities assisted G&E in the development of this report. They provided various types of data about their electric transmission systems.

Company Name	Province / State	Country
American Transmission Company	WI	USA
Bonneville Power Administration	OR	USA
British Columbia Transmission Corporation	BC	Canada
Hydro One Networks, Inc	ON	Canada
Hydro-Quebec	QC	Canada
New Brunswick Power Transmission Corporation	NB	Canada
Newfoundland and Labrador Hydro	NL	Canada

The support and guidance by Leon Kempner of BPA, Ali Afshar of Hydro One Networks, Janos Toth of BCTC, Asim Haldar of Nalcor Energy / NALH, John Williamson of NBPTC, Jean Claude Carriere and Maryse Lavoie of Hydro Quebec, and Adam Ramme of ATC, is greatly appreciated.

Additional support was provided by Transpower and Orion, electric power companies of New Zealand.

EXECUTIVE SUMMARY

Wind and ice loads on transmission towers have repeatedly damaged high voltage (115 kV and higher) transmission circuits in Canada and the United States. This has happened in events exceeding line design basis such as the ice storm of April 1993 that affected New Brunswick, as well as winter storms in Washington in 1999 and Idaho in 2010 that had nominal wind speeds well under the design basis, and yet still damaged towers. There are many other examples of damaged transmission towers in all U.S. states and Canadian provinces; as well as around the world including Australia, New Zealand and France.

All modern transmission towers in Canada and the USA have been designed for some level of wind and ice loads. There are various industry guidelines and codes that set rules and procedures for such design; these guidelines and codes have evolved over time, but it is rare (almost never) that older transmission towers are upgraded to the latest standards. Many transmission system owners customize the industry guidelines to reflect their own unique conditions. The wind and ice load conditions of coastal Florida are entirely different than in coastal Newfoundland, interior Quebec, interior Wisconsin or the Pacific coast. Reliability models need to be able to accommodate all these widely varying conditions.

A reliability model is developed that can be used by system operators to examine the reliability of each transmission tower in a transmission circuit. The model is geared to examining reliability for various types of loading conditions: user-entered scenario storms (either historical or forecast); or user-entered probabilistic storms (such as 50-year storm, 1,000 year storm, etc.). The wind model can be used for hurricanes along the Gulf Coast, Florida and eastern seaboard (USA and Canada), winter storms (anywhere in the USA or Canada). The model can be used for wind + ice storms (anywhere in the USA or Canada). The model can factor in downbursts from thunderstorms or high intensity winds from tornados, albeit in a simplified fashion if the user has site-specific knowledge, or lacking such knowledge, by increasing the uncertainty parameter for tower-specific wind speeds. The model can factor in local wind speed-ups due to topographical effects. The model allows the user to include tower-specific weaknesses such as locally weak soil conditions under saturated conditions, or construction defects, or other factors that were not reflected in the original design of the tower. The model allows spatial variation of wind and ice at different locations along the transmission line. The model accounts for apparent reliability increases (or decreases) due to as-installed horizontal spans that are shorter (or longer) than originally assumed for design. The model allows the user to enter age related factors, to accommodate decay (wood poles) or corrosion (steel lattice structures), and thus can allow the user to quantify reliability considering the level of ongoing maintenance.

The model is geared to be used by engineers familiar with transmission tower design issues. Calibration of the model to reflect local conditions is essential; if the model is used without calibration, the results may not be appropriate for the specific application being examined.

The model is developed to accommodate two levels of analyses: Level 1 (simplified, rapid) and Level 2 (somewhat more detail, requiring somewhat additional effort). Level 1 and Level 2 analyses should be adequate to examine transmission circuit reliability with reasonable accuracy for many applications. Neither the Level 1 or Level 2 analyses requires new structural analyses, quantification of line loads under all loading conditions; so the knowledgeable user can rapidly obtain rational reliability results. More detailed Level 3 refinements can always be made to further study site-specific conditions, such as computational fluid dynamics, nonlinear structural analyses, cable sag and tension forces, postcollapse dynamics, subsurface investigations, etc.; this report indicates where such Level 3 refinements might be suitable.

The model discussed in this report is used to study of three specific transmission lines that have been damaged by past storms: two examples are for high wind events (BPA 230 kV and 500 kV transmission lines), one example is for an ice loading event (New Brunswick 345 kV transmission line). The model can be set-up as an Excel-based application for simple circuit alignments. The model is incorporated into G&E's SERA software, available by license to Electric Utilities in Canada and the USA.

16 sensitivity analyses are performed for the two wind event case studies, to examine how the reliability of the model is affected by adjusted parameters such as tower fragility; wind speeds; topographic effects; tower capacity uncertainty; wind speed uncertainty; and truncation models.

5 sensitivity analyses are performed for the ice + coincident wind event case study, to examine how the reliability of the model is affected by adjusted parameters such as coincident wind speeds; topographic effects; construction defects; and truncation models.

A tower cascading model is presented that accounts for the potential for additional tower failures caused by collapse initiation of any tower. The cascade model uses as input the probability of failures from the tower collapse initiation models, as well as user-entered fragilities for zippering failure modes for each type of tower.

A benefit cost model is provided. Using nine return periods from 10 years to 10,000 years, the benefit cost model is run to examine the following question: is it economically worthwhile to retrofit an existing transmission line to substantially reduce the chance of cascade-type failures?

Using suitable parameters, these models are shown to have reasonable capability to forecast actual tower failures in past storms. It is recognized that calibration to past storms and tower performance is relatively straightforward, but extension of a calibrated model for different towers under different storm loads might not be as reliable. The user is cautioned that without suitable calibration and judgment, the model can provide unreliable forecasts.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

This report describes a model to evaluate the reliability of transmission lines under high wind and ice loading conditions.

1.1 Reliability Model

The model was developed using three example transmission lines: Satstop-to-Aberdeen-2 230 kV (Bonneville Power Administration, BPA); Hatwai-to-Dworshak 500 kV (BPA); and Line 3002 345 kV, New Brunswick Power Transmission Corporation. Each of these three lines suffered collapses of one or more towers due to wind, or wind + ice, in 1999, 2010 and 1993, respectively.

The model is provided in six Excel spreadsheets. Contact G&E Engineering Systems Inc. to obtain copies of these spreadsheets. As each of the example transmission lines has a different number of towers, and the reliability formulation for wind (alone) or wind + ice (combined) is slightly different, there are some differences in the Excel spreadsheets for each transmission line. The six Excel spreadsheets are as follows:

- Tower failure for Satstop-to-Aberdeen-2. Satsop-Aberdeen 4.xls
- Tower failure for Hatwai-to-Dworshak. Hatwai-Dworshak 0.xls; Hatwai-Dworshak 1.xls.
- Conversion of latitude-longitude into projected units. L3002 LatLong_km.xls
- Tower failure for Line 3002. Line 3002 0.xls
- Tower cascades and benefit cost analysis. Cascade 1.xls

The intent of the model is to examine various approaches that can be used to simulate line reliability due to wind or combinations of ice + concurrent wind loading. Each model might have "pros" and "cons" in terms of the level of effort needed to create the model, versus the quality of the results. Recognizing that different agencies will have different needs for these models, the model has the following features:

- Level 1 Model. The Level 1 Model can be used for very rapid loss estimation. Ideally, the end-user need spend perhaps only a few hours or so to collect data and run the model. The quality of the results should be useful, at least on a high-order level, in order to provide a first order estimate of the line reliability.

- Level 2 Model. The Level 2 Model includes some refinements to provide improved loss estimation. The refinements might include input additional inventory, hazard and fragility information as compared to the Level 1 Model, in order to develop more accurate results as compared to the Level 1 Model. The quality of the results should be more useful than those from the Level 1 Model, in order to provide an improved estimate of the line reliability. The Level 2 model uses the same Excel spreadsheets as the Level 1 model, but with refined data.

- Level 3 Model. The term "Level 3 Model" is used in this report to mean any model of more complexity than either the Level 1 or Level 2 models. We highlight what type of information might be developed in order to perform more detailed analysis. It is beyond the scope of this report to provide models to perform detailed Level 3 analyses.

1.2 Level 1 Model

We gear the Level 1 Model to rely on the minimum effort to establish inventory, hazards, fragility. For the examples in this report, we find that the Level 1 Model can somewhat reasonably estimate line reliability. We make this finding on the following observations:

- The Level 1 Model includes specific tower locations (latitude, longitude); ignores exposure, ignores topographic effects, uses wind or wind+ice fragility models for each style of tower.
- The Level 1 Model relies on nearby weather station data to establish likely loading on each tower and the line as a whole.
- The Level 1 Model provides transmission line reliability, from end to end, as well as the chance of failure of each specific tower.
- The Level 1 model can provide a reasonable first-order estimate of the line reliability, and can highlight the relatively weakest towers along the entire transmission line.

1.3 Level 2 Model

The Level 2 Model requires more effort to establish inventory, hazards, and fragility. The Level 2 Model can better estimate tower-specific reliability as well as line reliability. We make this finding on the following observations:

- The Level 2 Model includes local topographic effects for local wind speed-ups.
- The Level 2 Model includes site-specific factors to account for locally weak geologic conditions or local construction defects, that affect tower capacity to withstand high winds / winds+ice.
- The Level 2 Model can provide reasonable estimates of the line reliability, and individual tower reliability. The Level 2 Model, after calibration, provides a reasonable approach to select specific tower to upgrade to improve overall transmission circuit reliability.

1.4 Requirements

The models presented in this report require the user to have a license for Excel (from Microsoft). The databases discussed in this report require the user to have a license for Filemaker (from Claris). The models and databases should run identically using Windows XP, Vista, 7, 8, 10 or Mac OS X 10.6 to 12.3 operating systems. Contact G&E Engineering Systems Inc. if you are interested in obtaining the electronic models and databases.

To successfully apply the models in this report, it is assumed that the user has a firm understanding of reliability, the design of transmission towers and their conductors and appurtenances, the nature of wind storms, the nature of ice loading, and structural analysis. It is the end user's responsibility to select all parameters for the models and to confirm the adequacy of the results for the specific application at hand.

1.5 Abbreviations

ASCE	American Society of Civil Engineers
BCA	Benefit Cost Analysis
BCR	Benefit Cost Ratio
BPA	Bonneville Power Administration
CDL	Customer Day Lost
Cdn	Canadian
EF	Enhanced Fujita scale
F	Fujita scale
FEMA	Federal Emergency Management Agency
GCS	Geographical Coordinate System
GDP	Gross Domestic Product
GIS	geographical information system
GRP	Gross Regional Product
IBC	International Building Code
LADWP	Los Angeles Department of Water and Power
N	Number of circuits needed to provide reliable power.
NPV	Net Present Value
PNW	Pacific Northwest (Washington, Oregon)
RP	Return Period (in years)
RS	Rated Strength
SRTM30	Shuttle Radar Topographic Mission
UBC	Uniform Building Code
UTM	Universal Transverse Mercator

1.6 Engineering Abbreviations

Fa	Allowable working stress for steel, usually between 60% to 66% of Fy
Fy	Yield stress for steel
kV	kilovolt
kph	kilometers per hour
m	meter
mm	millimeter
m/s	meters per second
mph	miles per hour
MW	megawatt
N/m^2	Newton per meter squared
PGA	Peak Ground Acceleration
s	second
V	Volt, or Velocity
Vs30	Shear wave speed (m/s) in top 30 m of soil
Zo	Surface roughness, m

1.7 Conversions

This report presents results in customary units (feet, inches, etc.) and SI units (meters, etc.). The units are used interchangeably. The following provides conversions for the units used in this report.

1 inch	= 25.4 millimeter
1 millimeter	= 0.001 meter
1 meter	= 3.28 feet
1 foot	= 0.3048 meters
1 pound per cubic foot	= 16.018 kg/m^3
1 kilometer per hour (kph)	= 0.27778 m/s
1 knot	= 1 nautical mile per hour
1 knot	= 0.514444 m/s
1 knot	= 1.852 kph
1 foot/s	= 0.3048 m/s
1 mph	= 0.44704 m/s
1 mph	= 1.609 kph
1 millibar	= 0.029 inches of mercury, Hg

1 nautical mile (International) = 1852 m ~ one minute of arc of latitude along any meridian (north – south direction). This is the common measure in current use, worldwide. As the surface of the world does not have a constant radius, the actual length of a nautical mile varies from about 1842.9 m at the equator to about 1861.7 m at the poles.

Older maritime maps do not use the modern definition of a nautical mile (International). The following describes the older (now abandoned) definitions:
1 nautical mile (Admiralty) = 6,080 feet = 1853.184 m (abandoned in 1970)
1 nautical mile (USA) = 6,080.20 feet = 1853.248 m (abandoned in 1954)

Wind exposures. Unless otherwise noted, design wind speeds are commonly listed with respect to open terrain conditions commonly found at airports. In the USA, this is commonly called "Exposure C". In Canada, this same condition is commonly called "Exposure B". To avoid confusion, the term "Exposure C" as used in this report *always* refers to the common USA definition.

2.0 BACKGROUND

This report describes a series of Excel spreadsheets (collectively, the model) that can be used to examine the reliability of high voltage transmission towers under extreme wind, or ice with concurrent wind loads.

Two failure mechanisms are of primary interest:

- Failure of a first tower due to excessive loading, and
- Failure of adjacent towers due to cascading caused by the failure of the first tower. The terms "cascading" and "zippering" are used interchangeably.

These failure mechanisms can be initiated under several scenarios:

- Weather events (winter storms, wind storms, hurricanes, tornados, ice storms).
- Ongoing "age" related failures. The deterioration of metal (due to corrosion), insulators and other parts of transmission towers will lead to occasional failures of a tower. The age-related deterioration of wood poles is well documented. Ongoing maintenance activities to detect and repair age-related phenomena can eliminate or largely mitigate such deterioration.
- Accidental mechanical impacts. These might include aircraft hitting a tower from above, or motor vehicle impacts from below.
- Landslides (including those activated by intense rainfall or earthquake).
- Avalanches or snow creep.
- Earthquakes (including those induced by ground shaking, liquefaction, fault offset or landslide).
- Human-caused (terrorism).

This report addresses tower failures due to wind, or ice with concurrent wind loading.

2.1 Acceptable Reliability

From a reliability point of view, typical large-scale transmission system operators (serving more than 1,000,000 people) have certain amounts of reliability "built-in" to the network. Often, transmission system operators design their networks, and are allowed to charge for, under a "N-1" security rule.

- By "N-1", it is meant that the maximum load served by the transmission network can be sustained, with some margin, assuming any single transmission circuit is unavailable.

Maximum load might be summer-time air conditioning load (southern climates) or winter-time heating load (northern climates).

It is accepted practice in North America that all large-scale transmission system operators have the bulk of their networks able to perform assuming the "N-1" rule.

The reliability models presented in this report are geared to address potentials for a single scenario event (winter storm, tornado, hurricane) to result in impacted network reliability under N-2, N-3, … N-x situations. Since most networks are not designed for N-x situations ($x \geq 2$), such events will generally result in power outages, possibly with lengthy restoration times.

The design guidelines and codes for transmission towers now in use for most of North America set as a "minimum" the 50-year return period wind speed, commonly using a working stress design approach (also called "allowable stress" design approach). In areas prone to icing, a second load case is imposed that is a combined amount of radial ice with a coincident wind speed (usually 40 to 56 mph).

Newer codes now becoming available (ASCE 7-10 and later) have revised the wind design approach to explicitly recognize ultimate strength design provisions using a load factor design approach. Maps in ASCE 7-10 show wind speeds with 700 year and 1,700 year return periods, to be used with a load factor design approach.

Based on historical observation, it is recognized that not all transmission towers have to be "indestructible" in any kind of wind / ice storm event. To the extent that tower failures are "rare", coupled with network redundancy, it is acceptable to have some towers fail in infrequent storms. For example, BPA has an inventory of about 90,000 transmission towers; over the past 50 years, there have been about 200 tower failures due to wind, and a similar number due to ice. The infrequency of the tower damage, coupled with network reliability, has resulted in historical power outages which are not too long or too widespread as to have invoked great economic harm to the millions of people that live and work in the BPA service area. As a first-order estimate, a tower failure rate of 0.0005 per year (about 5 tower failures out of 100,000 towers, per year) seems acceptable to society.

This individual tower target reliability of 0.0005 has been observed to be reasonable over a long observation time (over 50 years) over a geographically wide area (Washington, Oregon, Idaho, parts of Montana and northern California). However, this is not the only measure of acceptable reliability. For example, should a single storm result in damage to 100 towers on 20 separate transmission lines, then widespread and lengthy power outages will occur, as BPA has capability of rapidly replacing only a few towers at a time. The "acceptable" rate of tower failure (circuit failure) in this case will depend upon several factors:

- In highly concentrated high economic activity areas (like most major metropolitan areas in Canada and the USA with metro population over 1,000,000 people). A single rare storm should not result in total blackout (due to transmission line failures) for more than a few hours, 90% of the customers should have power restored within 24 hours, nearly 100% within 3 days, maximum day (N security) load in 7 days, and maximum day (N-1 security) within 14 days.

- In moderately concentrated economic activity areas (like most cities in Canada and the USA with metropolitan population between 100,000 to 1,000,000 people). A single rare storm should not result in total blackout (due to transmission line failures) for more than 24 hours, 90% of the customers should have power restored within 48 hours, nearly 100% within 7 days, maximum day (N security) load in 14 days, and maximum day (N-1 security) within 28 days.

- In low concentration economic activity areas (like most rural areas and towns in Canada and the USA with metro population under 100,000 people). A single rare storm should not result in total blackout (due to transmission line failures) for more than 72 hours, 90% of the customers should have power restored within 7 days, nearly 100% within 14 days, maximum day (N security) load in 28 days, and maximum day (N-1 security) within 60 days.

In developing these target reliability goals for the transmission system, we implicitly assume that some damage also occurs to the low voltage sub-transmission (generally 60 kV to 70 kV) and distribution (generally 400 V to 33 kV) systems. In many parts of Canada and the USA, sub-transmission and distribution system are undergrounded. Underground cables are generally not vulnerable to wind and ice-storm damage; but when damaged, take a long time to repair. In areas of Canada and the USA with overhead systems, there will likely be concurrent damage to both the transmission, sub-transmission and distribution systems; and thus times to restore power to customers will depend on repair work to all these systems. Single ice storms have created widespread damage to many tens-of-thousands of wood distribution poles, which have resulted in long power outages (many weeks to many months) in rural areas. Because the grid, geography and repair crew capability of power utilities varies everywhere, there is no "single" set of reliability rules that can be established that would be cost effective everywhere in Canada and the USA.

Another important factor in establishing suitable reliability goals is the severity of a single storm, and its return period. For example, the ice storm of 1998 resulted in 1,000+ transmission tower failures and 30,000+ distribution pole failures, affecting wide areas of urbanized and rural Quebec, Ontario, New Brunswick and parts of New England. The thickness of ice accumulated on some transmission conductors approached 3 inches in this storm, which were probably in excess of the underlying design target; and hence the great number of tower and pole failures. This type of storm had previously not been considered for design, and the "return period" for 3 inches of radial glaze ice was considered "too extreme" and not often (if ever) observed in the region's previous 50 years (or so) of recorded history for ice accumulations. Clearly, had the original tower and pole design for the region used 3" of radial ice, then the widespread damage would have been largely avoided. However, the underlying issue is "at what cost"? In other words, is it worth the incremental extra cost to design a tower for extra thickness of ice? This type of question can be "solved" by using benefit-cost analyses (BCA) of the type presented in Section 7 of this report. One of the key assumptions in BCA is the return period for various size storms. If we had 1,000,000 years of well recorded history of prior ice storms, we could reliably establish the annual frequency of various types of storms. But, as we have barely 50 to 75 years of reliable history of past storms, we must be humble and accept that any mathematical model that extends 50 to 75 year trends to return periods like 1,000 to 10,000 to 100,000+ year events is open to challenge; more likely than not, such an extrapolation will be wrong.

Even so, we recognize from BCA that storms with return periods of 20 to 100 years are those most likely to have serious consequences to society that are cost-effective to mitigate; whereas the cost to mitigate for storms with return periods over 2,500 years is generally not cost effective. For extremely high consequence / high value facilities (such as commercial nuclear power plants), it is practical to simply design for the "worst" event possible, i.e., deterministic design. For example, in Arkansas it is prudent to design a nuclear containment building, as well as all its other safety related features, to withstand a direct hit by a EF-5 tornado, including wind and wind-borne debris. On the other hand, it does not make economic sense to design to this same "worst case" scenario for a nearby steel lattice transmission tower.

To summarize:

- We recommend that individual towers be designed to have a chance of failure due to wind or ice storms of about 0.0005 per year, in consideration of 50-year storms. This value *does not* factor in system-wide considerations such as redundancy, ability to make repairs, storm events with return periods much longer than 50 years, or local economic conditions. However, as a broad-brush target reliability goal, it appears to be economically sound. This rate of tower failure has been de-facto achieved over 50+ year periods over much of the west coast of the USA, with apparently no widespread and recurring customer complaints of extended outages. We presume this target to also be suitable for the hurricane-prone coast of the USA and eastern Canada; albeit to achieve this goal along the Gulf Coast (for example) will require design to higher wind speeds than along the Pacific Ocean coast of the USA and Canada.

- We recommend that transmission system operators consider long-return-period storms (or select a suitable scenario storm) to evaluate the single-storm damage to their entire network, and resulting power outages. The target restoration times listed above should be customized for the actual region, such that the benefits (reduced future losses) outweigh the costs (higher initial construction costs) for towers and the network as a whole. Such a vulnerability analysis should factor in the entire system, the various storms it might be exposed to, the ability to make repairs (with mutual aid), and a combination of possible mitigation (upgraded towers, more redundant network) and emergency response strategies that can be implemented to reduce the severity of impacts, in a cost effective manner.

3.0 SAMPLE ANALYSIS 1: WIND EVENT

In Section 3, we present a reliability analysis for a 230 kV transmission line that failed in 1999 in a high wind event. This line is owned by the Bonneville Power Administration (BPA).

BPA has had a number of towers collapse due to high wind and ice loads. For the period from 1950 to 2009, a total of 161 towers were damaged due to high winds, and another 61 towers damaged due to ice loads (Figure 3-1). Of these, there have been a few cases where the damage was a broken conductor (under 3% of the time). In some cases the damage was dropping the conductor due to failure of the porcelain insulators.

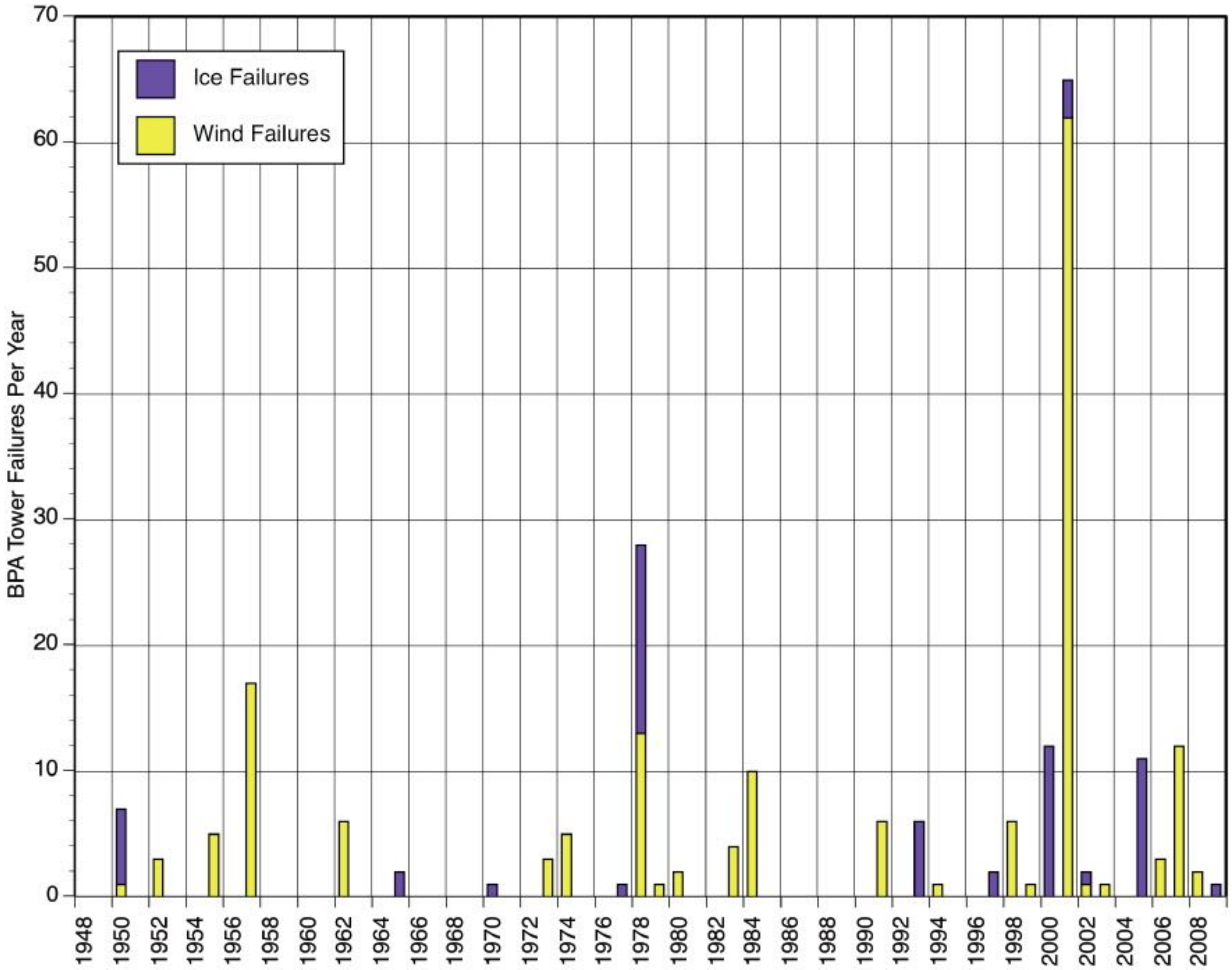

Figure 3-1 Reported Tower Failures (BPA), 1948 - 2009

The data in Figure 3-1 is thought to be nearly complete for steel lattice structures. The data in Figure 3-1 may omit data for damaged wood pole-type structures if the construction crews effected repairs without specifically reporting said repairs to the BPA transmission group. Most of the damage in Figure 3-1 was for towers on 115 kV to 500 kV lines, and could be due to wind- or ice-plus-wind-induced overloads on specific above ground members or foundation failures; in a few cases, the damage was caused by wind-induced fallen trees. Damage due to landslides, snow slides, vehicle impact, airplane impacts or other non-weather related causes is not included in Figure 3-1. The large number of wind-tower (pole) failures in 2001 was on a line near Pasco, where high winds failed 61 poles on one wood pole line (wood poles snapped near their base).

Figure 3-1 does not distinguish between towers that were damaged due to high wind and those adjacent towers that were damaged due to the unbalanced conductor loads that occur once the first tower collapses. With the exception of the one wood-pole event in 2001, it is apparent that unbalanced-load tower failures have not been common (in a few initiations, perhaps 1 or 2 adjacent towers collapsed) in the BPA system. This has not been the case for other high-voltage power utilities. For example, observational failures of Hydro-Quebec's 735 kV towers in the 1998 ice storm suggests that the failure of a single tower along a long circuit led to rapid collapse of every adjacent tower due to unbalanced conductor loads until the pull-down towers reached a dead end tower, or a tower with very short spans (near substations). Elsewhere in the USA and worldwide, it has not been uncommon to see 5, 6 or 7 adjacent towers collapsed in a wind event. It is speculative whether BPA's design practices are sufficiently different from those at other high-voltage transmission utilities such that BPA's steel lattice tower configurations are inherently more resistant to unbalanced line loads, but the BPA empirical evidence of the past 60 years would certainly imply that this is the case.

3.1 Description of the Line

A sample transmission circuit is used in order to perform reliability analyses for wind loads. The single 230 kV circuit is called the Satsop – Aberdeen # 2 line. This line is located in Washington State, near the Pacific Ocean.

This circuit is owned and operated by the Bonneville Power Administration. Detailed information about this circuit may not be released to third parties without the prior written approval by BPA.

This 230 kV circuit is located in western Washington State. Figure 3-2 highlights a portion of the 230 kV transmission network in western Washington State, with the Satsop – Aberdeen # 2 line going between the two noted substations (large triangles).

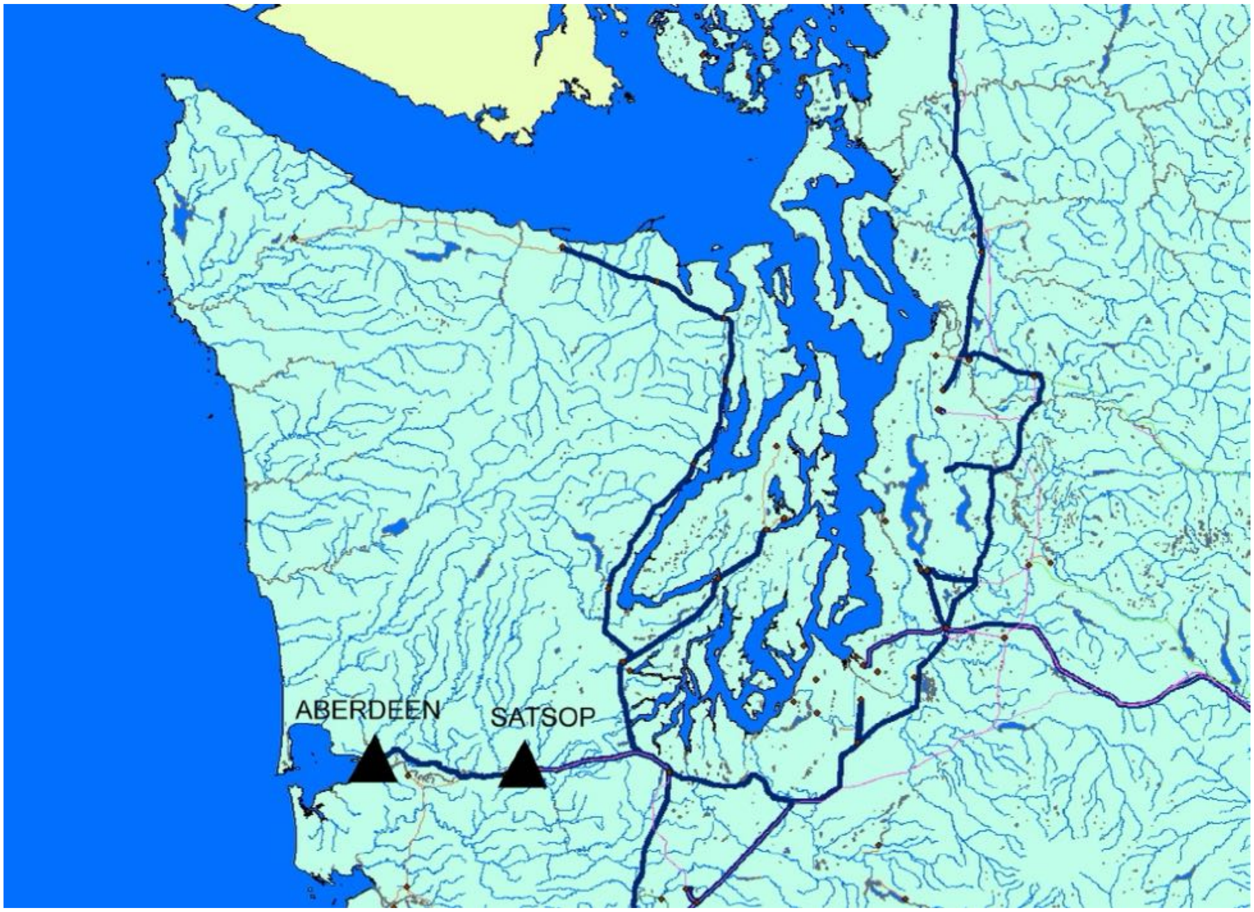

Figure 3-2 Satsop-Aberdeen Circuit Regional Map

On February 23, 1999, at about 11:53 pm, tower 18/2 (mile 18, structure 2) failed during a high wind event. Failure was characterized as a footing failure. The BPA tower type (internal BPA nomenclature) was type 22L (light suspension tower).

On December 2, 2007, six more towers failed on this line due to high winds. These structures were all light tangent suspension towers.

Data about this line includes:

- 230 kV
- Circuit Abbreviation: SATS-ABER-2
- Entire circuit length (substation to substation) = 20.48 miles = 33.3 km.
- Total of 96 towers from Satsop to Aberdeen substations.
- Average horizontal distance between towers = 1,188 feet.

- Conductor type = Drake (diameter 1.108 inches, 1.094 pounds per foot, rated strength (RS) 31,500 pounds), 1 conductor per phase. Approximate tensions:40% of RS i.e.12,600 pounds (wind only); 50% of RS i.e.15,750 pounds (rime plus concurrent wind); 60% of RS i.e. 18,900 pounds (glaze ice plus concurrent wind)

- One ground wire, 7 No. 8 Alumoweld, diameter = 0.386 inches, 0.262 pounds per foot, rated strength 15,900 pounds. Approximate tensions: 40% of RS i.e. 6,360 pounds (wind only); 50% of RS i.e. 7,950 pounds (rime plus concurrent wind);60% of RS i.e. 9,540 pounds (glaze ice plus concurrent wind).

- Original construction era: 1960s. To our knowledge (as of 2011), there were no site-specific adjustments considered in the 1960s-era design to account for local geologic conditions for the foundations of the towers.

- Ruling span = 1,460 feet.

Figure 3-3 shows each individual single-circuit tower along the circuit. The Aberdeen substation is shown as the large triangle on the left, the Satsop substation is the large triangle on the right, and the towers between the substations are the small open circles. The delivery of power for this line begins at the Satsop substation in the east and goes west to Aberdeen. Essentially all power in Aberdeen is imported from generation power plants far to the east, and all the power goes through the Satsop substation. Along the alignment near Satsop, the circuit is south of the Hoquiam river; then crosses the river; and then continues to Aberdeen on the north side of the Hoquiam river. Prevailing winds come from the southwest, blowing toward the northeast.

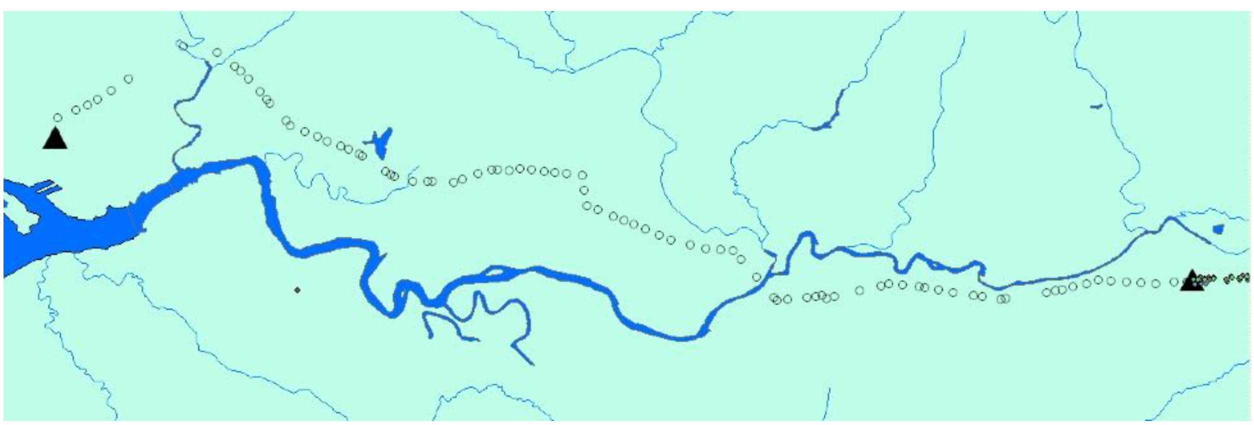

Figure 3-3 Satsop-Aberdeen Circuit Local Map (Circle = Tower Location)

Figure 3-4 shows the circuit (line) and individual single-circuit towers near Aberdeen, labeled with the type of tower. The tower that failed in the 1999 wind storm is enclosed by the large circle. As is not uncommon when using data from GIS systems, there appears to be some towers missing (either side of tower type B2) in the database; but actually, at these locations the towers are double circuit:

- Most of the towers (91) along the Aberdeen-Satsop-2 line carry just the single circuit.

- Five of the towers along the Aberdeen-Satsop-2 line also carry the Aberdeen-Satsop-3 line.

When developing the reliability model, one needs to keep track of the "span ahead" and "span-behind" distances. This is relatively easy to do when one selects the towers along the line from a database that includes only single-circuit towers. However, this can become somewhat tricky when there are towers that carry two (or more) circuits.

For a reliability analysis done on a circuit-level (from substation to substation or from substation to tap), one must track the reliability of each individual tower in the circuit, whether it holds one or two (or more) circuits.

Figure 3-4 Satsop-Aberdeen Circuit Local Map (Text = Tower Type)

Figure 3-5 shows the general land use patterns and topography near the 22L tower that failed. The tower that failed is located alongside a hill ("X" in the figure below), at a tower base elevation of about 280 feet. Figures 3-6 and 3-7 show the topography near the failed tower (Figure 3-6) and the adjacent non-failed tower (Figure 3-7) across Bear Gulch Road.

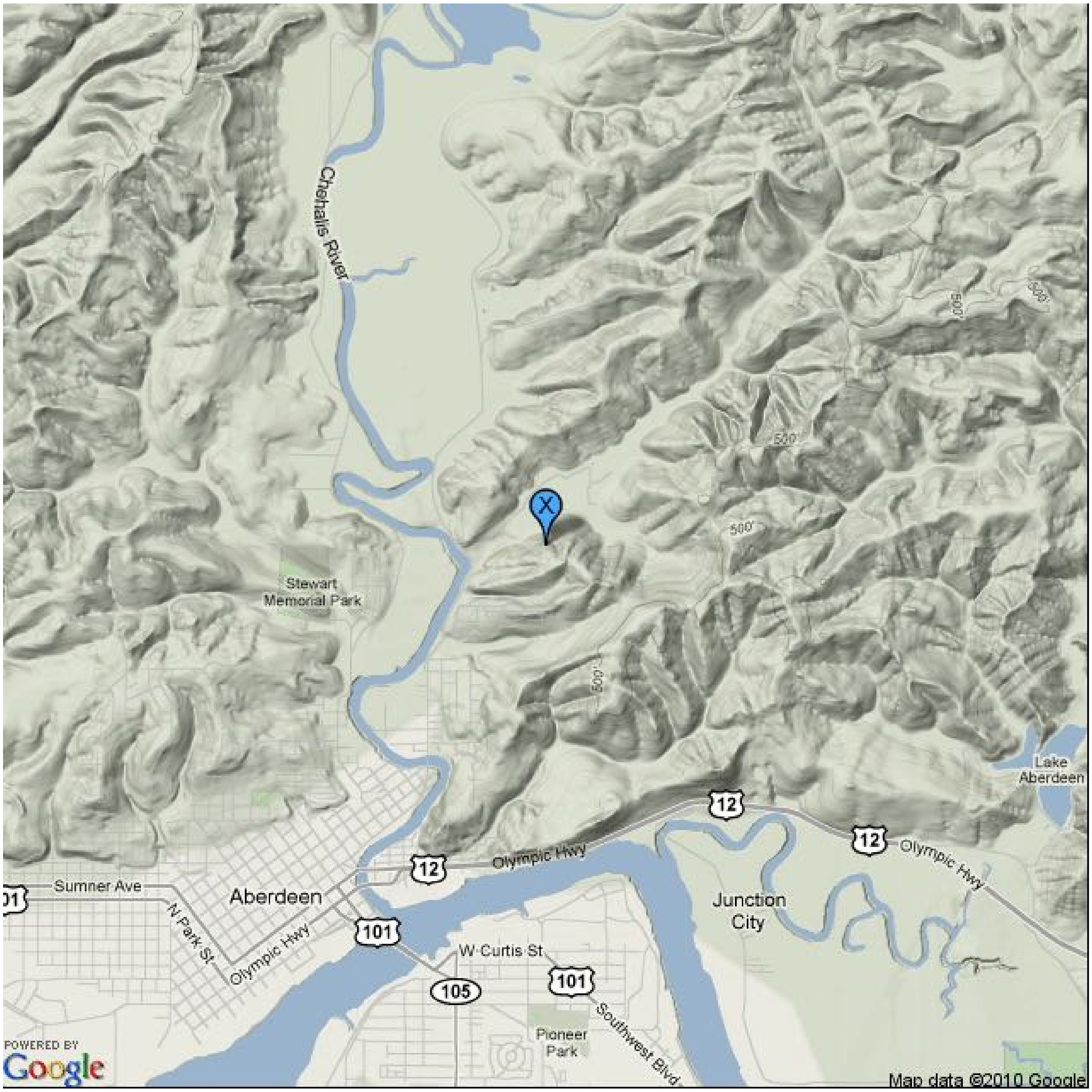

Figure 3-5 Local Topographic Map (X = Tower 18/2, Type 22L, with Failure)

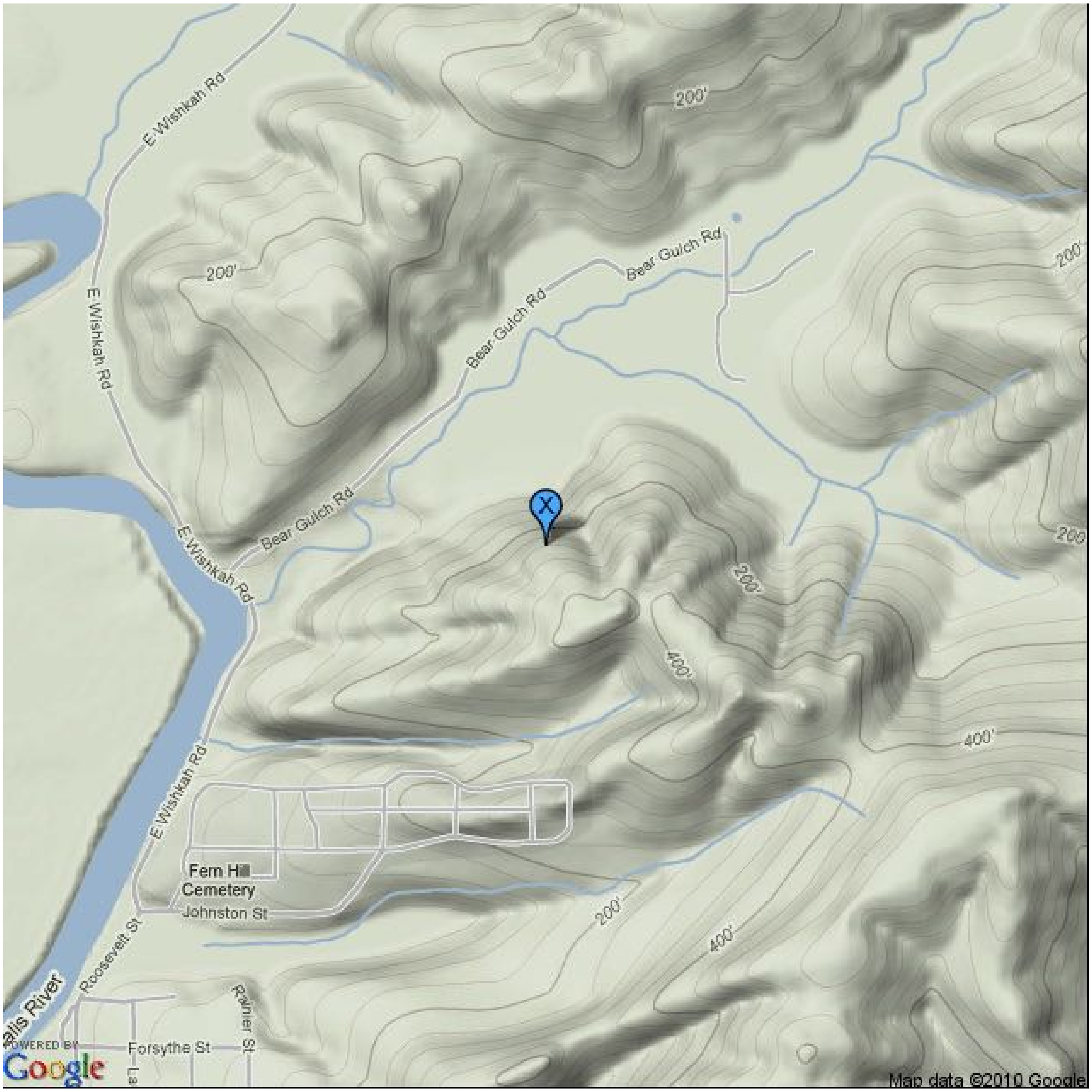

Figure 3-6 Local Topographic Map (X = Tower 18/2, Type 22L, with Failure)

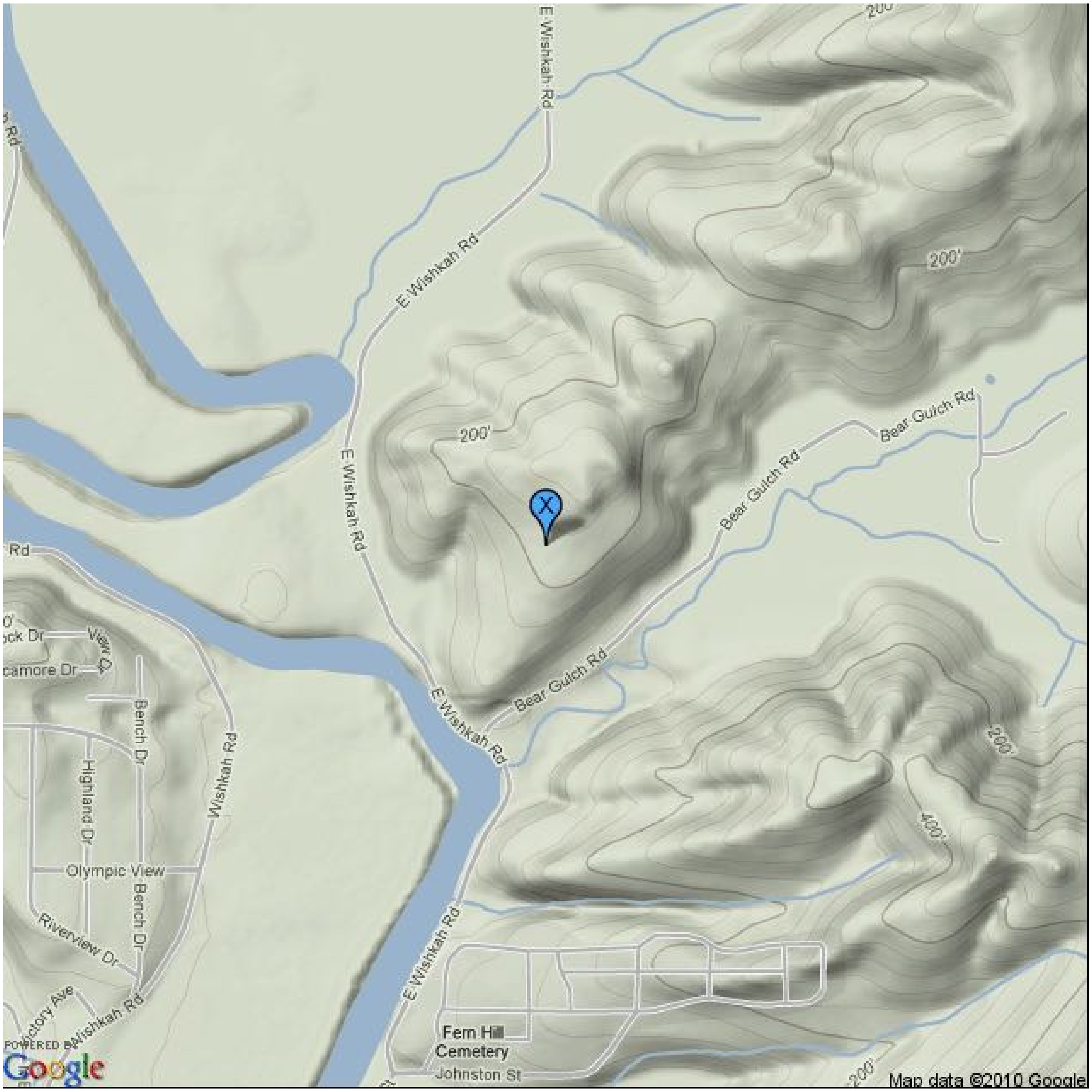

Figure 3-7 Local Topographic Map (X = Adjacent Tower 18/3, Type 2H1, No Failure)

3.2 Description of the Towers

Using the BPA naming convention, there are many types of tower designs used in this circuit. In order to perform a reliability analysis, we need to quantify the original design capacity of each tower design, and then adjust the design-capacity to reflect actual field conditions.

- 22L: 46 towers. Light tangent suspension tower.
- 2B: 2 Heavy tangent suspension tower
- 2D: 1 Angle tower

- 2D1; 1 Angle tower
- 2H1: 6. Heavy tangent suspension tower
- 2H2: 2. Heavy tangent suspension tower
- 2K: 7 Angle tower
- 2N: 1 Angle tower
- 2P: 16 Angle tower
- 2S2: 2. Standard tangent suspension tower
- 2SI: 3. Standard tangent suspension tower
- 3B1: 1. Heavy tangent suspension tower (double circuit)
- 4A1: 1. Standard tangent suspension tower (with ground wire)
- 4D1: 1. Dead-end tower (with ground wire)
- 5B1: 1. Heavy tangent suspension tower (double circuit)
- 5D2: 1. Medium dead-end tower (double circuit)
- SSDE (dead end towers, on at each substation): 2

The original design for the above 2-series towers (tower type starts with the number "2") was for a Drake conductor (diameter 1.108 inches). The original design for the 4- and 5-series was for a Pheasant conductor (diameter 1.345 inches). The actual line uses Drake conductor, one conductor per phase, with no ground wire. Therefore, when doing the reliability analysis, an adjustment for the 4- and 5-series towers needs to be done, to reflect that in-situ they are loaded with a smaller conductor and no ground wire.

An important "book-keeping" aspect of doing the reliability analysis is to have a perfect, errorless naming system. This can be problematic as the actual GIS databases may contain many kinds of errors. For example, in the above dataset, there were two "B2" towers in the BPA database, which needed to be corrected to "2B", so that the computer database functions can select the proper fragility function. Similarly, type "2SI" was likely mis-typed and should have been "2S1". We then uncovered that the fragility capacity results for tower type 2B were mis-typed into the model as 1150 feet; although the standard span for type 2B is 2,000 feet; we discovered this after running the reliability analysis that constantly showed the type 2B towers to be grossly overloaded; after correcting the fragility database, the reliability results were logical.

One of the points in the preceding paragraph is that when dealing with large datasets, there is great potential for error, especially if the raw dataset comes from a GIS database that has not been proof-checked for the intended reliability analysis purpose. While this type of error can be readily uncovered when performing the reliability analysis on one circuit at a time, it becomes a challenge to verify the analyses when performed on 100,000 towers at a time.

From these topographical figures, we can observe that structures 18/2 (type 22L, Figures 3-6) and 18-3 (type 2H1, Figure 3-7) are exposed to potentially high wind loads on the conductors that span across Bear Gulch Road. The following are some of the tower characteristics (W is the wind speed at which the type of tower at the design span has F.S. = 1.0):

- 18/2 (failed tower). Type 22L (light tangent). Database ID: 14179. Span ahead to 18/3: 1,895 feet. Span behind to 18/1: 615 feet. Actual tributary span = 1,255 feet. Design span = 1,200 feet. Angle change: 0.02 degrees. W = 92 mph.

- 18/3 (intact tower). Type 2H1 (heavy tangent). Span behind to 18/2: 1,895 feet. Span ahead (to 19/1): 858 feet. Design span: 1,450 feet. Actual tributary span = 1,376 feet. Angle change: 0.11 degrees. W = 100 mph.

Figure 3-8 shows the actual horizontal spans for each tower along the circuit. Tower 14179 is highlighted: it collapsed in the actual 1999 storm event.

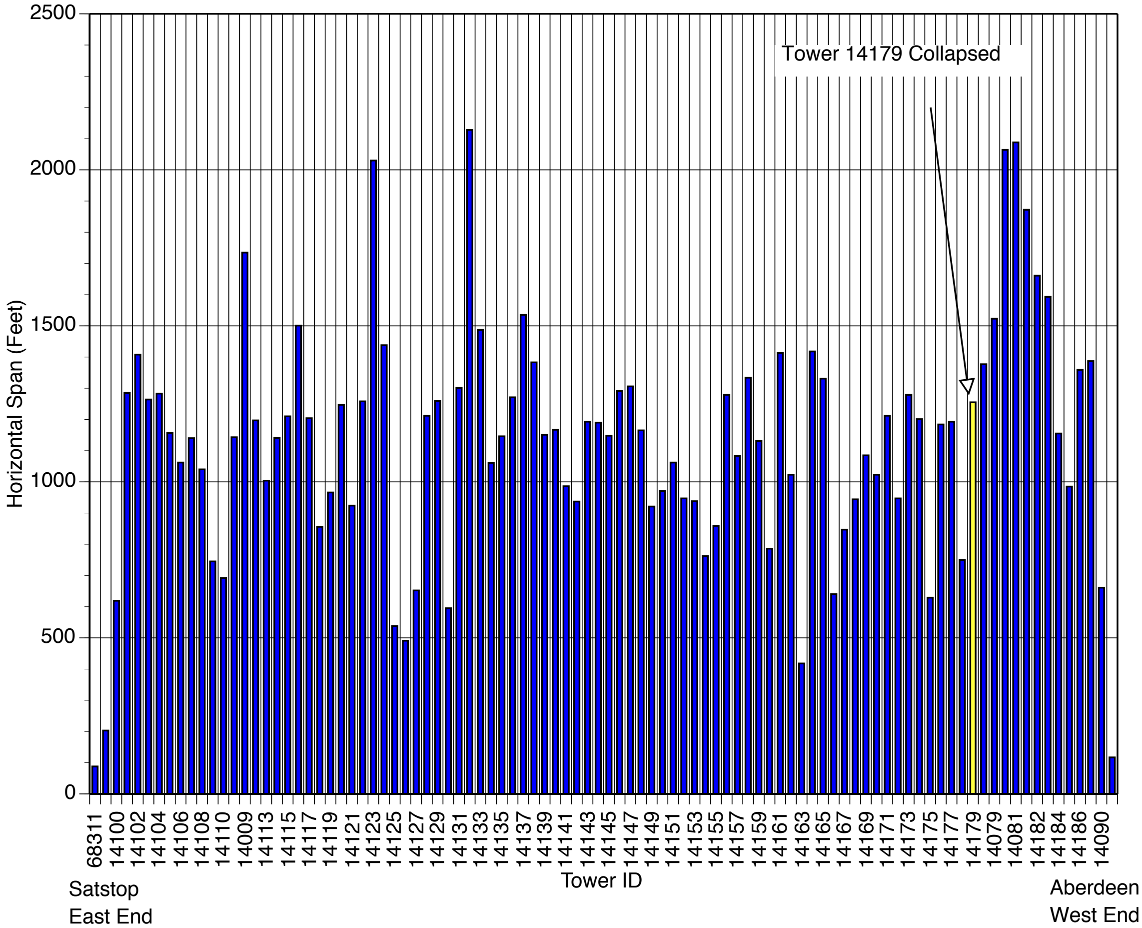

Figure 3-8 Satsop-Aberdeen Circuit Spans

Figure 3-9 shows the tower types and actual line angles along the circuit (same order as in Figure 3-8). As can be seen, the actual line angles for tower type 22L (light suspension tangent) are nearly always exactly 0°, although one type 22L (14168) has a line angle of 2.18 degrees. As type 22L was designed for no line angle loads, even a 2.18 degree line angle change will introduce some un-planned for loads on the tower due to conductor tensions. For this particular tower (14168), this is offset somewhat as the actual horizontal span is about 944 feet, quite a bit lower than the design span of 1,200 feet.

Figure 3-9 also shows that the selection of the type types are matched to BPA's standard tower type classifications based on line angle. Dead-end towers are designed for full unbalanced conductor loads. Line angles are the horizontal angles along the line (always assumed at 90° for type SSDE). Tangent and Angle towers are designed for one broken conductor. The following describes the "standard" BPA design practices, although in practice there are hundreds of minor variations. The lettering system denotes: no ground wire / with ground wire. The numbering system that precedes the letter description denotes voltage and number of circuits per tower (single circuit: 0, 2, 4, 6, 8 = 115 kV, 230 kV, 287 kV, 345 kV, 500 kV). (double circuit: 1, 3, 5, 7, 9 = 115 kV, 230 kV, 287 kV, 345 kV, 500 kV)

The following lists the common design assumptions for various 230 kV BPA towers built since 1980.

- Light Suspension Tangent (2L, M). Design line angle = 0°; span = 1150 feet.
- Standard Suspension Tangent (2S, A). Design line angle = 0 to 3°; span = 1425 feet.
- Heavy Suspension Tangent (H, 2B). Design line angle = 0 to 6°; span = 1450 feet.
- Angle (C). Design line angle = 0 to 15°; span = 1200 feet.
- Light Dead End (PL, DL). Design line angle = 0 to 30°.
- Medium Dead End (P, 2D). Design line angle = 30 to 60°; span = 1550 feet.
- Heavy Dead End (N, G). Design line angle = 60 to 90°; span = 2150 feet.
- SSDE Strain Dead End. Design line angle = 90° (down) plus 15° horizontal.

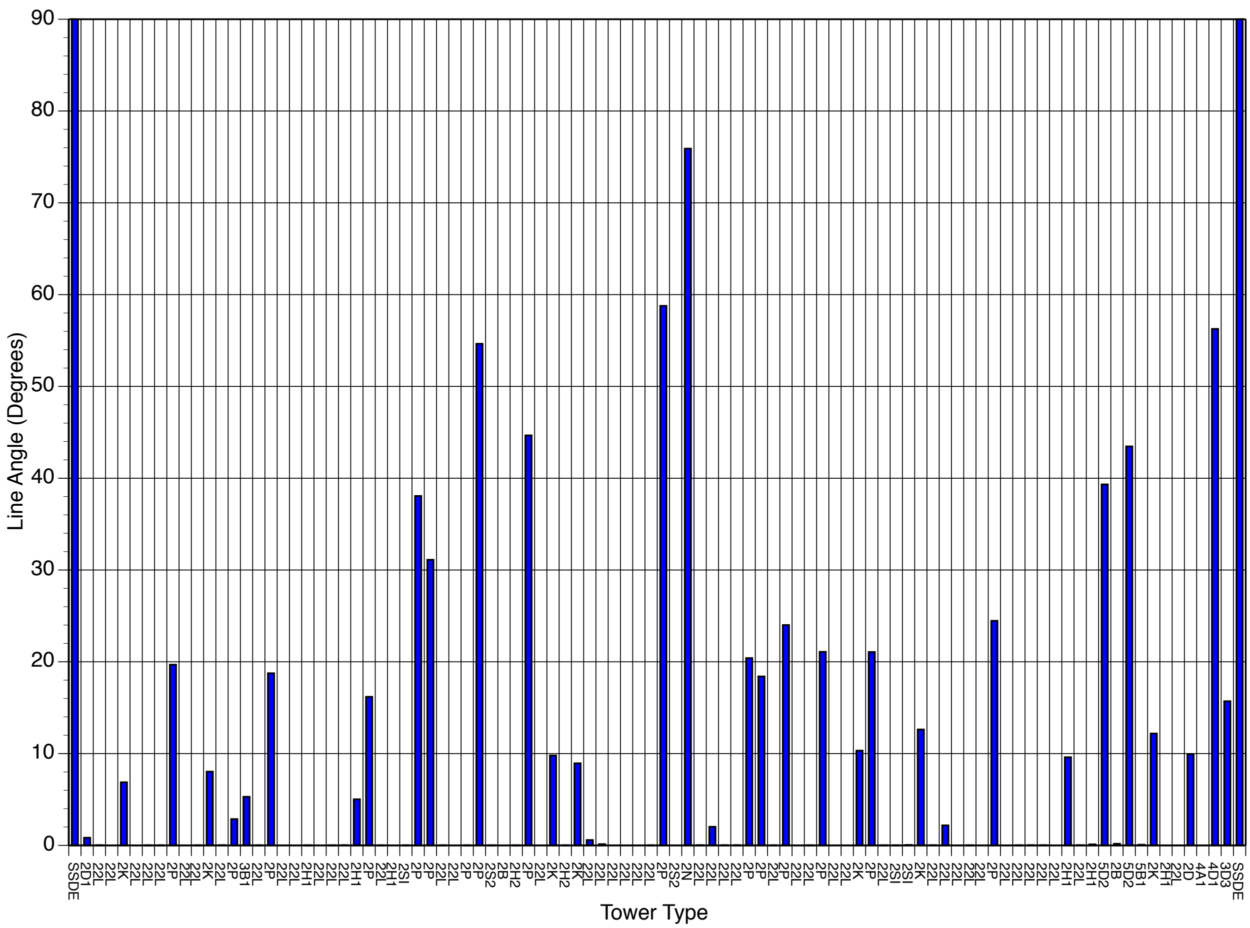

Figure 3-9 Satsop-Aberdeen Tower Types and Line Angles

Although the numbering / naming system of towers is mundane, it is a necessary and critical aspect of doing the reliability analysis.

3.3 Strength Capacity and Fragility of the Towers

For purpose of the reliability analyses presented herein, we need to establish two basic parameters for each type of tower:

- Strength Capacity. This is the common engineering calculation of the strength (like in pounds) of a tower to resist a particular external wind (or ice) load (mph, or inches + concurrent mph). This is the type of calculation that most transmission tower designers are familiar with.

- Fragility. This is the expression that relates the chance of tower collapse (initiation) given the occurrence of an external wind or ice + concurrent wind. Section 6 extends the fragility for tower collapse initiation to also include cascading (zippering) failures.

Section 3.3.1 describes the computation of the Strength Capacity. Section 3.3.2 describes the conversion of the Strength Capacity into a Fragility.

3.3.1 Strength Capacity of Towers

There are three primary weather load cases considered in the BPA design approach for steel lattice towers:

- Case A: Extreme Wind. Also referred to as 0-25-0, meaning 0 inches of ice, 25 psf wind force, ambient temperature 0°F

- Case B1: Rime Ice plus concurrent wind. Also referred to as 2R-4-0, meaning 2 inches of rime ice, 4 psf wind force (40 mph), 0°F

- Case B2: Glaze Ice plus concurrent wind. Also referred to as 1-8-0, meaning 1 inches of glaze ice, 8 psf wind force (56.5 mph), 0°F

While steel lattice towers are designed to nominal yield level for these three load cases, most towers should usually be able to withstand somewhat larger loads than these levels. A variety of other load cases are used to design BPA towers (dead load, stringing loads, unbalanced ice loads, etc.), but it is felt that these other load cases do not usually control for tower failures under extreme wind or wind+ice loading cases; for other reliability analysis situations where any of these other load cases might control, this assumption would need to be relaxed.

The strength capacity of a BPA tower is based on the principles of engineering mechanics, to relate the wind (Case A) or ice + wind (Cases B1, B2) combination that leads to nominal overload of the tower. For purposes of reliability analyses, the strengths (usually expressed in units like pounds, pound per square inch, etc.) are converted to external loading values, either the 3-second gust speed (mph) or ice thickness (inches, with concurrent wind speed, mph). Externally applied forces applied by winds are commonly assumed to vary as the square of the wind speed. Therefore, a 10% increase in wind speed corresponds to a 21% increase in wind force applied to the tower, as $1.21 F = (1.1 * V)^2$.

The optimal approach to establish the strength capacity and fragility of each individual type of steel lattice tower would be a combination of full scale testing and nonlinear structural analyses. This level of effort is costly and time consuming. In modern times, it is common to use 3-dimensional structural analysis programs to evaluate the forces in each member of a tower; and one can use large geometry nonlinear analyses to get the cable tension loads for any particular load condition, sag, slack and operating temperature. For purposes of this report, this level of complexity is commonly not warranted for Level 1 or Level 2 analyses; but certainly may be suitable for retrofit design of specific towers or design of new towers.

For Level 1 and Level 2 analyses, we established the capacities of the towers along the Satsop – Aberdeen line in a simplified manner. The intent of this simplified manner was to produce rational nominal ultimate strength capacities of common styles of steel lattice towers.

To determine the capacities of each tower type, the static load overturning calculation was compared to the design footing capacity; alternative tower failure modes, such as buckling, shear, or tensile failure, were not considered. The tower capacity was determined using the maximum wind (Case A), rime ice (Case B1), or glaze ice (Case B2) that could be applied on each tower before overturning about one of the most eccentric legs would occur.

Required user input to establish the design wind speed (or wind speed + ice thickness) of each type of tower required the following inputs:

- Transverse Span; Vertical Span.
- Conductor Details: Outer Diameter; Unit Weight; Number of conductors; Rated strength and maximum working tension (MWT); Horizontal distance from most-eccentric leg (i.e. overturning leg) to attachment point of each conductor; Height from overturning leg to attachment point of each conductor.
- Ground wire Details: Outer Diameter; Unit Weight; Number of ground wires; Rated Strength and maximum working tension Horizontal distance from overturning leg to attachment point of each ground wire; Height from overturning leg to attachment point of each ground wire.
- Tower Height; Tower Weight; Tower Wind Area; Distance from overturning leg to farthest-opposing leg; Distance from overturning leg to second-farthest leg; Distance from overturning leg to closest leg; Maximum Uplift of a tower leg foundation.
- Minimum Horizontal Line Angle; Maximum Horizontal Line Angle.

Application:

- Conductor bundles were accounted for by multiplying the number of phases on the tower by the number of sub-conductors, and then inputting required data for each sub-conductor.

- Due to the varied combinations of body extensions and leg extensions possible for each tower type, it was decided that a 20-ft body extension and 25-ft leg extension, or whichever combination was closest, should be used to provide an average tower configuration.

A series of assumptions were made and applied in the static overturning calculation in order to establish the design-base wind speeds for the analyses presented in Sections 3 and 4 of this report.

- A transverse span of 1200 ft was used, if its value could not be determined otherwise.

- A vertical span of 130% of the transverse span was used, if its value could not be determined otherwise (the scalar was decided by, conservatively, reducing the ratio of all found transverse spans to their corresponding vertical spans).

- The total tower pressure due to wind has a centroid occurring at 2/3 of the tower height.

- The tower weight has a centroid occurring at 1/2 the distance from the overturning leg to farthest leg.

- The weight of insulators, ice on the tower, and hardware was not included.

- Uplift varies directly with the distance from overturning leg, reaching the inputted maximum uplift at the maximum distance from the overturning leg.

- Applied line angle was assumed to be the average of the minimum and maximum line angles
- Tower wind speed acting is 125% of conductor wind speed. In other words, the applied wind pressure was taken as 1.25 * 1.25 = 1.5625 times the wind pressure applied to the conductors.

- For extreme wind (Case A), the longitudinal tension is 40% of the RS of the conductor; for extreme rime ice with wind (Case B1), it is 50% of the RS; for extreme glaze ice with wind (Case B2), it is 60% of the RS These percentages are based on a statistical study.

Clearly, any of the above assumptions could be relaxed, to use tower-specific computations. Such refinements might be done as part of any Level 3 analysis.

Using these assumptions, an implicit determination of maximum wind speed, maximum rime ice, and maximum glaze ice were determined. The methodology can be found below. The sign convention used is illustrated for a generic tower in Figure 3-10.

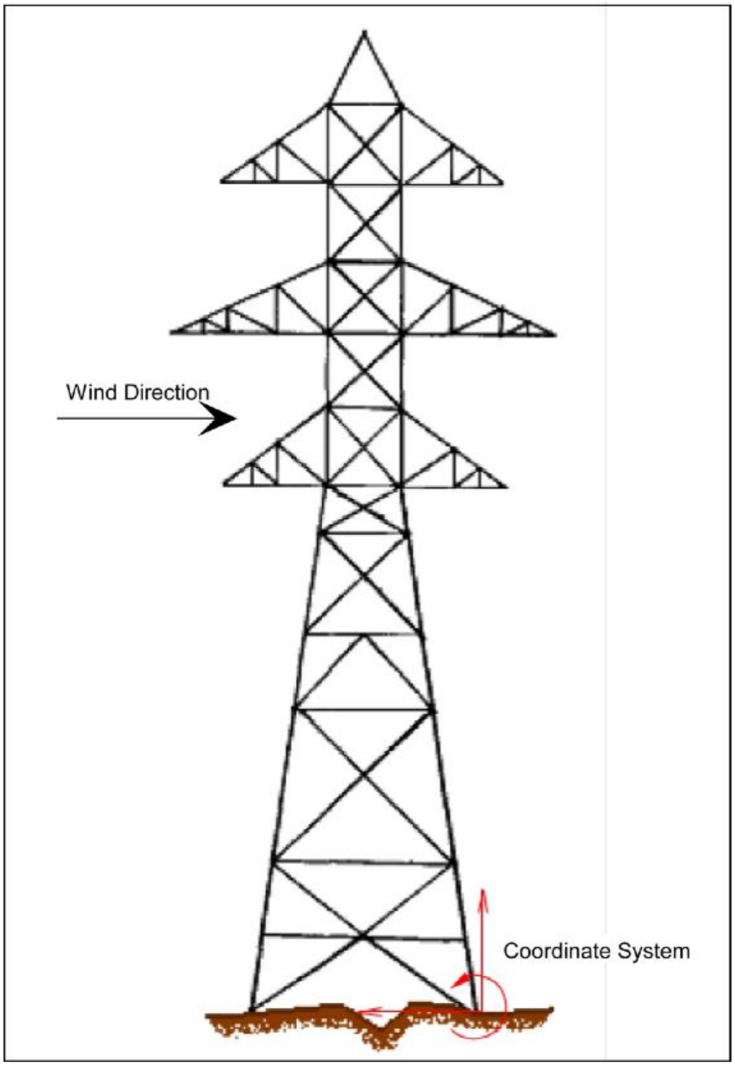

Figure 3-10 Generic Tower Coordinate System

Maximum Wind (Case A)

Summing moments about the overturning leg, as shown in Figure 3-10 based on the direction of the wind, due to the following forces:

- Wind loads on conductor and ground wire
- Wind load on tower
- Transverse load due to line angle (assumed to occur in the same direction as wind, to provide a more conservative estimate)
- Conductor and ground wire weights
- Tower weight
- Uplift

$$0 = \sum M = -\left(F_{Tower} \cdot \frac{2}{3} h_{Tower} + F_{Cond} \sum_{Conductors} h_i + F_{GW} \sum_{GroundWires} h_j + T \sum_{Cond,GW} h_k \right)$$
$$+ W_{Tower} \cdot \frac{1}{2} d_{Leg1} + W_{Cond} \sum_{Conductors} d_i + W_{GW} \sum_{GroundWire} d_j + \sum_{Legs} U_k d_k$$

where

F_{Tower} = wind load on tower, pounds

h_{Tower} = tower height, feet

F_{Cond} = wind load per conductor, pounds

F_{GW} = wind load per ground wire, pounds

h_i = vertical attachment height on tower, feet

d_{Leg1} = distance from overturning leg to farthest leg, feet

W_{Tower} = vertical weight of tower, pounds

W_{Tower} = vertical weight per conductor, pounds

W_{Tower} = vertical weight per ground wire, pounds

d_i = horizontal distance to attachment point or leg, feet

U_i = uplift capacity per leg, pounds

By ASCE 74:

$F_{Tower} = 0.004V^2 \cdot A$ (this presumes the wind speed is 25% faster on the tower than on the conductor or ground wire)

$F_{Cond} = 0.00256V^2 \cdot D_{Cond} \cdot TSpan$

$F_{GW} = 0.00256V^2 \cdot D_{GW} \cdot TSpan$

where

A = tower area orthogonal to wind direction, feet^2

V = conductor wind speed, mph

D_{Cond} = conductor diameter, feet

D_{GW} = ground wire diameter, feet

$TSpan$ = transverse span length, feet

Additionally,

$$W_{Cond} = \lambda_{Cond} \cdot VSpan$$

$$W_{GW} = \lambda_{GW} \cdot VSpan$$

where

λ_{Cond} = conductor unit weight, plf

λ_{GW} = ground wire unit weight, plf

$VSpan$ = vertical span, ft

Based on the aforementioned assumption that line angle tension contributes a transverse load:

$$T = \phi \cdot MWT \cdot \sin\left(\theta_{Avg} / 2 \right)$$

where

ϕ = 40% (case A)

θ_{Avg} = average line angle

$$= \frac{\theta_{Min} + \theta_{Max}}{2}$$

Based on the aforementioned assumption that uplift varies linearly with distance:

$$U_i = \frac{d_{Leg\_i}}{d_{Leg1}} U_{Max}$$

where

d_{Leg_i} = distance from overturning leg to i^{th} leg, ft

d_{Leg1} = distance from overturning leg to farthest leg, ft

d_{Leg_i} = maximum uplift, lbs

Subsequent substitution and algebraic manipulation gives:

$$V = \sqrt{\frac{\frac{1}{2} \cdot W_{Tower} \cdot d_{Leg1} + W_{Cond} \sum\limits_{Conductors} d_i + W_{GW} \sum\limits_{GroundWire} d_j + \sum\limits_{Legs} U_k d_k - T \sum\limits_{Cond,GW} h_k}{0.004 \cdot \frac{2}{3} \cdot A \cdot h_{Tower} + 0.00256 \cdot TSpan \cdot \left(D_{Cond} \sum\limits_{Conductors} h_i + D_{GW} \sum\limits_{GroundWires} h_j \right)}}$$

where V is the maximum wind speed in mph that the conductor can sustain before the tower overturns.

Note: in applying the above model, the tower weight and its moment contribution to resistance to overturning ($\frac{1}{2} \cdot W_{Tower} \cdot d_{Leg1}$) is excluded in calculating the maximum wind speed. This exclusion provides a more conservative (lower) maximum wind speed for the tower capacity, as the tower weight would contribute to resistance to overturning.

Maximum Ice (Rime Ice Case B1, Glaze Ice Case B2)

If one makes the assumption about the maximum concurrent wind speed, then the ice radius becomes the only unknown variable in calculating maximum rime ice and maximum glaze ice, the same procedure is used, with the differences in known values shown below in Table 3-1:

	Variable used	Rime ice with wind	Glaze ice with wind
Longitudinal tension (*% of MWT*)	ϕ	50	60
Design wind speed (*mph*)	v	40	56.6
Ice density (*pcf*)	ρ	15	57

Table 3-1. Model Parameters

Summing moments about the overturning leg gives:

$$0 = \sum M = -\left(F_{Tower} \cdot \frac{2}{3} h_{Tower} + F_{Cond} \sum_{Conductors} h_i + F_{GW} \sum_{GroundWires} h_j + T \sum_{Cond,GW} h_k \right.$$
$$\left. + W_{Tower} \cdot \frac{1}{2} d_{Leg1} + W_{Cond} \sum_{Conductors} d_i + W_{GW} \sum_{GroundWire} d_j + \sum_{Legs} U_k d_k \right)$$

By ASCE 74:

$$F_{Cond} = 0.00256V^2 \cdot \left(D_{Cond} + 2 \cdot R_{Ice}\right) \cdot TSpan$$

$$F_{GW} = 0.00256V^2 \cdot \left(D_{GW} + 2 \cdot R_{Ice}\right) \cdot TSpan$$

where

R_{Ice} = Ice radius, ft

Note that for a tower with no ground wire, the above strength calculation applies the ice radius to the ground wire. While this is inaccurate for towers without ground wires, it provides a more conservative estimate (lower ice thickness) of maximum ice thickness.

Additionally,

$$W_{Cond} = \left\{\lambda_{Cond} + \rho\pi\left(D_{Cond} \cdot R_{Ice} + R_{Ice}^2\right)\right\} \cdot VSpan$$

$$W_{GW} = \left\{\lambda_{GW} + \rho\pi\left(D_{GW} \cdot R_{Ice} + R_{Ice}^2\right)\right\} \cdot VSpan$$

Subsequent substitution and algebraic manipulation gives:

$$a \cdot R_{Ice}^2 + b \cdot R_{Ice} + c = 0$$

where

$$a = \rho\pi \cdot VSpan \cdot \sum_{Cond,GW} d_i$$

$$b = -0.00256 \cdot TSpan \cdot V^2 \sum_{Cond,GW} h_i \cdot 2 + \rho\pi \cdot VSpan \cdot \left[D_{Cond} \sum_{Conductors} d_j + D_{GW} \sum_{GroundWire} d_k \right]$$

$$c = -0.004 \cdot \tfrac{2}{3} \cdot A \cdot h_{Tower} - 0.00256 \cdot TSpan \cdot \left(D_{Cond} \sum_{Conductors} h_i + D_{GW} \sum_{GroundWire} h_j \right)$$

$$-T \sum_{Cond,GW} h_k + \tfrac{1}{2} \cdot W_{Tower} \cdot d_{Leg1} + VSpan \cdot \left(\lambda_{Cond} \sum_{Conductors} d_m + \lambda_{GW} \sum_{GroundWire} d_n \right) + \sum_{Legs} U_q d_q$$

Therefore,

$$R_{Ice} = \begin{cases} 12 \cdot \dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}, \text{for } a \neq 0 \\ -12 \cdot \dfrac{c}{b}, \text{for } a = 0 \end{cases}$$

where R_{Ice} is the maximum radius of ice, in inches, required to cause overturning.

The value for *U*, uplift capacity, is obtained using typical foundation design assumptions, trying to conservatively assume the varying geotechnical conditions that might commonly be encountered: for example, for granular materials, a density of 90 pounds per cubic foot might be assumed, with a friction angle of 30 degrees. As will be discussed later in this report, the true capacity of soils at actual tower installation locations might be weaker than the assumption; and this might lead to tower failures at wind speeds much below those assumed in design.

We checked the validity of the above simplified procedures versus use of a 3-dimensional structural analysis program, for four types of towers. Detailed 3D models were developed for four types of towers. The V value for the wind case from the above simplified procedure was applied to the 3D models. Each member in the tower was stress checked, with the stress ratio (stress ratio = applied force / force needed to yield or buckle the member) for the most critical member listed in Table 3-2. It is seen that the stress ratios from finite element analysis using actual member properties and all members is in reasonable agreement with the simplified approach.

Tower Type	Wind Speed Vsimple, MPH from Simplified Procedure	Stress Ratio at Vsimple	Accuracy, on wind speed, Vsimple
28M	135	0.949	-3%
18M	135	1.056	+3%
08B	150	0.988	-1%
5A	131	0.953	-2%

Table 3-2. Accuracy of Tower Capacity Values, Case A

From these four examples, we see that the computed value for V might be too high or too low by a few percent (as much as ±3% in these four examples). These results provide sufficient confirmation that the program developed gives reasonable preliminary level estimates of ultimate wind, rime ice, and glaze ice capacities.

3.3.2 Fragility of Towers

Table 3-2 is an excellent example of one of two types of error terms in reliability analyses:

- Errors due to uncertainty (Table 3-2).
- Errors due to randomness.

The errors due to uncertainty can be removed by doing more-and-more accurate analyses; as for example, by doing 3D structural analyses instead of the rather simplistic overturning moment calculation presented in this section. These types of uncertainties can also include: knowledge of actual Fy for each piece of steel, rather than using minimum specified values; inclusion of actual joint geometry rather than assuming "in-line" joints; consideration of the actual surface roughness on conductors rather than using a simple "span factor"; consideration of actual soil strength and cohesion properties at each tower leg, rather than simply assuming some lower bound but common capacity.

The errors due to randomness cannot be readily overcome by doing more precise calculations. For example, we do not ever really know the time-varying wind velocities at a point along a conductor; nor how the velocities vary along the conductor; even if we instrumented a conductor to obtain such variations in a single storm, we do not currently have the knowledge to forecast what they might be in the next storm.

For purposes of the extreme wind Reliability analyses, we express the extreme load applied to the tower as a wind velocity (V), and we wish to compare this to the tower fragility capacity.

Once the strength capacity is determined (Section 3.3.1), how does one assign the fragility level? We suggest using a truncated 4-parameter fragility model ($\hat{V}$, β_u, β_r, $D/C_{cut-off}$):

$\hat{V}$, the wind velocity that will result in collapse of 50% of similar towers (excluding cascade failures)

$\beta = \sqrt{\beta_u^2 + \beta_r^2}$, where β is the lognormal standard deviation of the variance in $\hat{V}$, which is he vector sum of the standard deviations for items that are uncertain and items that are random.

$D/C_{cut-off}$. To avoid spurious results, we truncate (cut-off) the distribution at a lower bound level. D repsents the demand on the tower, and C represents the capacity of the tower. For example, the chance of tower failure under applied wind speed = D = 10 mph, for any tower built to ASCE 74 with C = 100 mph, is, by observation, 0. Any continuous distribution (including the lognormal) will always return a chance of failure at D/C = 0.10; but we will want to avoid such spurious results.

The lognormal distribution shape is selected for two primary reasons:

- Because it is mathematically convenient. For any value of applied wind speed V (which, by definition, is always positive) the lognormal distribution will always provide a positive cumulative density function, no matter what value is set for . For example, if a normal distribution were taken, typical value of (standard deviation) would allow that there is a finite cumulative probability that the tower will collapse when exposed to V = 0 mph; clearly a nonsensical result.

- Because failures in the world tends to have lognormal distributions. Still, as of 2011, we do not have enough data for tower failures in extreme storm events to test whether this is in fact true for transmission towers.

In developing the reliability model, we observe that full scale tower tests have shown, than on average, is about 15% higher than V(strength capacity). In other words, if V(strength capacity) is 100 mph, then 50% of similarly-designed and installed towers should fail at 115 mph. This is the same as saying that the tower fails at a load (pounds or psf) that is 132.25% of the than the design load.

Where does this extra 32.25% come from? It is a combination of the conservatisms built into the elastically-based design; coupled with some post-yield capacity of a typical tower to accept more load before it actually collapses.

3.3.3 Strength Capacity of Towers – Simplified

Section 3.3.1 provides a way to compute the strength capacity of a tower, of one knows quite a large number of geometry and mass input variables, including the dimensions of the tower base, the weight of the tower, etc. In Section 3.3.3, we present an even more simplified way to compute the strength capacity of a tower, in particular when one already knows the answer to one of the three load cases (A – Wind, B1 – Rime + concurrent wind; B2 – Glaze + concurrent wind).

We perform this simplified calculation for tower type 22L, the tower that failed (circled tower in Figure 3-4). Tower type 22L is used commonly in the BPA system, with about 1,305 installations, system-wide. The original design for this type of tower assumed a 1,200-foot horizontal span.

We estimated the design strength capacity of this tower based on one of three load cases assuming a 1.602 inch diameter conductor (one conductor per bundle) with no ground wire. By "strength capacity", we mean that the highest-loaded primary load-carrying element has a F.S. = 1.0, whether in tension, shear or compression.

- Transverse wind, 92 mph on a 1200-foot span.
- Rime ice: 1.99 inches with 40 mph concurrent wind.
- Glaze ice: 0.81 inches with 56.6 mph concurrent wind.

Assuming a 0 degree line angle, the lateral load applied to the tower from the wind / rime / glaze ice loads on the conductors is as follows:

$$W = 0.00256 * L_{Tspan} * SF * V^2\left[n_{conductors} * OD_{conductor} + n_{ground\ \ wire} * OD_{ground\ \ wire}\right]$$

$$R = .00256 * L_{Tspan} * SF * V_{Rime}{}^2\left[n_{conductors} * \left(OD_{conductor} + 2 * t_{Rime}\right) + n_{groundwire} * \left(OD_{groundwire} + 2 * t_{Rime}\right)\right]$$

$$G = .00256 * L_{Tspan} * SF * V_{Glaze}^{2}\left[n_{conductors} * \left(OD_{conductor} + 2 * t_{Glaze}\right) + n_{groundwire} * \left(OD_{groundwire} + 2 * t_{Glaze}\right)\right]$$

where L(Tspan) is the horizontal span between towers (feet); SF = span factor = 0.7 for spans greater than 900 feet[1]; n = number of conductors (or ground wires); OD = outside diameter of conductor (or ground wire), in feet, V = wind speed for the Wind load case in miles per hour, Vrime = wind speed for the Rime Ice load case in mph; Vglaze = wind speed for the Glaze Ice load case in mph; W = transverse load on tower due to the Wind load case, pounds; R = transverse load on tower due to the Rime load case, pounds; G = transverse load on tower due to the Glaze load case, pounds.

To get the actual overturning moment would involve a similar calculation, but also includes the height above ground for each conductor / ground wire; plus the loads applied directly on the tower itself. For purposes of the reliability analyses, we believe that the total lateral load on the tower is a good first order proxy for the overturning moment; and is much simpler to calculate in a Level 1 or Level 2 analysis (does not require knowledge of cable heights, and ignores the load on the tower itself which is usually a small portion of the total overturning load).

- 22L. Original Design (1960s). Say it was originally designed for applied loads assuming V = 92 mph (Case A); or V = 40 mph, t = 1.46 inches (Case B1); or V = 56 mph, t = 0.78 inches (Case B2). Then, by applying the above calculations, we get the applied force on the tower as:
- Wind. W = 6,143 pounds. (input V = 92 mph)
- Rime. R = 4,510 pounds. (input V = 40 mph, t = 1.46 inches)
- Glaze: G = 5,907 pounds. (input V = 56 mph, t = 0.78 inches)

Since the three original load cases result in somewhat varying lateral loads on the tower, for this case we assume that the tower design was controlled by the Wind-only load case (92 mph). At this design wind speed, the most critical element in the tower has a factor of safety of 1.0 (elastic limit). For example, if the critical element is yielding of a main steel angle at Fy = 36 ksi, then the F.S. = 1.0 when the computed stress in this element is 36 ksi. Similarly, if the critical element is buckling, then at 92 mph the critical compression member is at its nominal buckling limit (with F.S. = 1.0).

For the Rime (Case B1) and Glaze (Case B2) loading cases, the resulting lateral force (4,510 or 5,907 pounds) is lower than the Wind-only lateral force (6,143 pounds). Therefore, as a first order approximation, we re-compute the required rime-ice thickness (and glaze ice thickness) needed to reach a lateral load of 6,143 pounds for those load cases. This first order approximation assumes that the concurrent wind speed in these ice-loading conditions is "fixed", and the "free" variable is the thickness of the accumulated ice. Clearly, this approximation could be relaxed in a Level 3 analysis, to reflect actual scenario storm event (actual V, t values might be used for a Level 3 analysis).

- 22L Calibrated Design (1960s)
- Design span 1,200 feet, 1.350" diameter conductor, no ground wire
- Wind. 6,143 pounds. (V = 92 mph)
- Rime. 6,143 pounds. (V = 40 mph, t = 1.99 inches)

[1] The Span factor of 0.7 is an approximate method to estimate the lateral load transferred from the conductor or ground wire to the tower. For longer (more than 900 feet) lengths between towers, the common design approximation is to multiply an assumed uniformly-applied load by 0.7; for shorter lengths between towers, the span factor is assumed to be 1.0. In practice, wind and ice loading is not constant either spatially or temporally.

- Glaze: 6,143 pounds. (V = 56 mph, t = 0.81 inches)

Similarly, the calibrated design for the other tangent tower types are as follows:

- 2S Calibrated Design (1960s)
- Design span 1,425 feet, 1.108" diameter conductor, no ground wire
- Wind. 6,084 pounds. (V = 92.74 mph)
- Rime. 6,084 pounds. (V = 40 mph, t = 1.69 inches)
- Glaze: 6,084 pounds. (V = 56 mph, t = 0.70 inches)

- 2H Calibrated Design (1960s)
- Design span 1,450 feet, 1.108" diameter conductor, no ground wire
- Wind. 7,198 pounds. (V = 100.0 mph)
- Rime. 7,198 pounds. (V = 40 mph, t = 1.64 inches)
- Glaze: 7,198 pounds. (V = 56 mph, t = 0.62 inches)

For towers with line angles, the lateral load from conductor tension onto the tower needs to be added into the analysis. For each conductor, the transverse tower load is estimated as follows

$$T_{wind} = Rated\ \ Strength * 2 * n_{conductor} * 0.3 * \sin(\theta/2)$$

$$T_{Rime} = Rated\ \ Strength * 2 * n_{conductor} * 0.4 * \sin(\theta/2)$$

$$T_{glaze} = Rated\ \ Strength * 2 * n_{conductor} * 0.5 * \sin(\theta/2)$$

where θ = the line angle (design); Rated Strength = rated strength (pounds) of the conductor.

- 2P Original Design (1960s) (60° Line angle)
- Design span 1,550 feet, 1.108" diameter conductor, no ground wire
- Wind. 21,975 pounds. Tension 28,350 pounds. Total 50,325 pounds (V = 169 mph)
- Rime. 3,875 pounds. Tension 37,800 pounds. Total 41,675 pounds (V = 40 mph, t = 1.19 inches)
- Glaze: 5,268 pounds. Tension 47,250 pounds. Total 52,518 pounds (V = 56 mph, t = 0.63 inches)

For this case, the maximum transverse load on a tower type 2P is 52,518 pounds, controlled by the glaze ice condition. We can then back-calculate the required wind speeds (rime or glaze ice thicknesses) needed to reach the same total transverse load on the tower, resulting in the following Calibrated Design for type 2P:

- 2P Calibrated Design (1960s) (60° Line angle)
- Design span 1,550 feet, 1.108" diameter conductor, no ground wire
- Wind. Total 52,518 pounds (V = 172.64 mph)
- Rime. Total 52,518 pounds (V = 40 mph, t = 1.50 inches)
- Glaze: Total 52,518 pounds (V = 56 mph, t = 0.63 inches)

In comparing the designs for 22L, 2S, 2H and 2P, we see that the line angle tower has de-facto a much higher wind load capacity than the tangent towers. This is not surprising, as the angle tower 2P is designed for a 60° angle change, which implied a substantial unbalanced lateral load on the tower due to cable tension.

The actual capacity of an angle tower 2P in the field will be greater (higher than Vwind, Trime, Tglaze) if any of the following conditions are actually used in the field:

- Tspan is shorter than the design span
- Line angle is less than 60°
- The actual conductor (for bundled conductors, the fewer conductors) has a smaller diameter / lower rated strength than the design conductor
- The actual ground wire (for multiple ground wires, the fewer ground wires) has a smaller diameter / lower rated strength than the design ground wire

The actual capacity of an angle tower 2P in the field will be lower (lower Vwind, Trime, Tglaze) if any of the following conditions are actually used in the field:

- Tspan is longer than the design span
- Line angle is more than 60°
- The actual conductor (for bundled conductors, the fewer conductors) has a larger diameter / higher rated strength than the design conductor
- The actual ground wire (for multiple ground wires, the fewer ground wires) has a larger diameter / higher rated strength than the design ground wire

3.4 Cascading Assumptions

A complete analysis of the failure of the Satsop – Aberdeen #2 line should take into account the effects of cascading / zippering after the initial tower failure. In the actual 1999 wind storm, only a single tower failed, and there was no cascading effect. Section 6 of this report examines the cascading issue in detail. So, while cascading failures can often be very important, Section 3 of this report concentrates only on the initial failure mechanism. For the reader's interest, and for purposes of presenting a complete picture of the design for this line, the following paragraphs describe the potential for cascading.

Should one tower collapse, then there is a chance the initial collapse will cascade (zipper) to adjacent towers. The potential for zippering, for this line, is as follows:

- Light tangent suspension tower: 50%
- Standard or heavy tangent suspension tower = 20%
- Light angle tower: 5%
- Heavy angle tower: 1%
- Full dead End tower (type SSDE): 0%

These percentages reflect the expert judgment of the transmission tower designer, which in part is based on field observations, as well as the design basis for each tower. For the towers in this line, all the tangent suspension towers were designed to accommodate failure of a single conductor, with the standard and heavy suspension towers incorporating a larger factor of safety than for the light tower.

There is some evidence to state that in a transmission line with many identical tangent structures, that the collapse of one structure imposes a certain amount of energy (via the broken conductors) to the adjacent tower. As each tower falls over, the energy used in the collapse process is subtracted from the energy available from the broken conductors to fail the next tower. In other words, a "zippering" circuit should eventually self-arrest. For purposes of the evaluation in Section 3, we ignore this phenomena; see Section 6 for an example with zippering.

3.5 Reliability Model Analysis Spreadsheet

The reliability analysis for the Satsop – Aberdeen #2 circuit is performed using an Excel spreadsheet. (Contact G&E Engineering Systems Inc. if you are interested in the Excel spreadsheets). The Excel file, Satsop-Aberdeen 4.xls, includes all the computations. We ran a total of 11 cases to examine how variation in the different variables in the model affects the outcome.

Before discussing the findings for each of the 11 cases, we first present the Excel model. Print-outs of the Excel model are not include in the report; the user may find it helpful to print out portions of the spreadsheets, or view them on a monitor, when reading this section. The model (file Satsop-Aberdeen 4.xls) has two types of sheets:

- Sheets 1 through 11. These represent the computation of circuit reliability under 11 different sets of assumptions. All of these 11 sheets (named Case 1, Case 2, … Case 11) rely of data in the Fragility sheet.
- Sheet 12. This sheet contains the fragility data for different types of towers (19 types in this model). If the user changes the assumptions in Sheet 12, the results in all of Sheets 1 through 11 will change.
- None of the cells in any sheet are locked. Color coding of cells was done for convenience, and has no bearing on the results. Not all the columns include data required for the analysis, and the user can remove these columns without affecting the results.

Some of the input data for the Excel file (Satsop-Aberdeen 4.xls) was selected out of a relational database that includes about 100,000 towers. Most large transmission system operators will have databases with all the towers, with various attributes for each tower. Some of the input data is not essential to the analysis, but is retained in the Excel file. Sections 3.5.1 and 3.5.2 describe all the data in the spreadsheet, and how it is used.

3.5.1 Sheet Case 1

The following describes the rows and columns in the Sheet for Case 1. It is recommended that the user print out the Excel sheet to help interpret the descriptions. These descriptions also apply to the Sheets for Cases 2 through 11.

- Row 1. This is the header row. **ROW 1 is for Information only.**

- Rows 2-97. These are the rows with the computations for each of the 96 towers in the circuit. **These rows contain the computational reliability model.**
- Rows 98-108. Various summations of the data for all 96 towers above. **These rows contain summary results of the computational reliability model.**

- Column A. objectID. This is a unique number that identifies the tower from a database. The actual value has no affect on the analysis. **COLUMN A is for Information only.**
- Column B. TWR-Type. This is the Tower Type that identifies the generic type of tower from the database. For example, SSDE (sub-station dead end), 22L (light suspension tower), etc. **COLUMN B is for Information only.**
- Column C. Revised Tower Type. Column C is the same as Column B, except that the user corrected a few of the tower types from the database. For example, tower 14132 (row 37) was listed as a "B2" in the database, but in fact is a "2B". **COLUMN C is INPUT** to the model.
- Column D. Structure Number. For a few towers, the common tower number system used by linemen (and as commonly painted onto the tower structure in the field) is included. For example, row 84 shows "18/2", meaning tower at mile 18, structure 2. This is the tower that actually collapsed in the 1999 wind storm. **COLUMN D is for Information only.**
- Columns E, F, G, H. Primary_TowerID_Ahead, Primary_TowerID_Behind, Secondary_TowerID_Ahead, Secondary_TowerID_Behind. These four columns list the TowerIDs for the adjacent towers ahead and behind of the tower on each row. In the database, each tower has latitude and longitude ordinates, as well as projection of those ordinates into eastings and northings (feet or meters) using a local projection system. In order for the Excel model to work, the horizontal distances between towers must be known. The user must use various GIS or other software to make the conversions from latitude / longitude into eastings / northings for this Excel spreadsheet to work. **COLUMNS E, F, G, H are for Information only.**
- Column I, J. These two columns show the horizontal distances between the tower on each row and the connected tower ahead and behind it. For this particular line, most of the towers are single-circuit towers; but a few are double circuit towers. The tracking of the ahead- and behind-spans to adjacent towers is done outside of this Excel spreadsheet. **COLUMNs I and J are INPUT** to the model.
- Column K. Angle (degrees). This column shows the horizontal angle of the conductors at the tower on each row. This angle is computed outside of Excel, and considers the connected tower ahead and behind it. Dead end towers are assigned 90 degrees by default. The tracking of the ahead- and behind-angles to adjacent towers is done outside of this Excel spreadsheet. **COLUMN K may be INPUT** to a refined model that factors in line loads, but is not used in the models described in this report.
- Columns L, M, N, O. These are the eastings, northings, latitude and longitude for the tower. These are input from the database. The model does not use this information, but these values are useful for making plots. The eastings and northings columns could be used to make to-scale plots within Excel. The latitude and longitude columns could be used to make maps using GIS software. The Projection and Geographic coordinate systems are not noted in the Excel file, but the user must have these if these locations are to be input into a GIS mapping system. There are no "default" coordinate systems used in the USA; some utilities use datum's from 1927, others from 1983; some use state plane projection systems in feet; others use UTM in meters. The choice of coordinate systems is largely governed by the style

of maps to be made, and a system that is useful for California will be highly distorted for Alaska; to there is no "single" coordinate system that is ideal for every transmission company in Canada and the USA. In fact, most utilities will want to use the coordinate system specific to their own utility, as that greatly helps making maps using pre-developed map layers, etc. **COLUMNS L, M, N, O are for Information only.**

- Column P. Slope_15x15 Pct (100% = 45 degrees). For this particular model, we pre-computed the slope of the terrain at each tower location, for a grid of 150 meters x 150 meters (with the tower in the middle of this plan-view box). The data is presented in percent (100% = 45°). Many GIS software programs use percent as the common way to list slope; and it is important that the end user know what the slope means (percentage of angle). The value of slope can be used as a proxy to indicate whether the tower is hilly or mountainous terrain. The slopes listed in this spreadsheet were developed using the SRTM mission, that computed X, Y, Z coordinates of the earth's surface at about a 10 meter horizontal grid area. Thus, each 10m x 10m box can have a slope. We do not list the slope for the specific 10m x 10m box underneath the tower, as there can be "flat spots" in mountainous terrain, and these local "flat spots" are not good indicators as to the general slope of the nearby terrain. Depending on what the end purpose of the slope is, the user might want to either omit this column (for example, this might be suitable in very flat terrain in Florida), or compute slope (as a proxy for topographic effects) in a different way. It is beyond the scope of this report to provide the models to compute topographic terrain. Case 1 does not use slope, but more refined models (Cases 5 to 11) do use the slope parameter. **COLUMN P is used for some cases as a proxy for topographical issues.**
- Column Q Actual Span. This is the horizontal span for this tower, defined as half the distance to the adjacent towers. **COLUMN Q is INPUT** to the model.
- Column R. Standard CAPACITY, Adjusted, wind, mph. This if the Fragility value (50% failure level) wind speed to initiate failure of this tower. For Row 2, the SSDE has a 221 mph value (very, very high). The Excel spreadsheet obtains this from the Fragility sheet, by sending over the value in C2 (tower type). The user must create the fragility table. The default Excel model includes a fragility table with 19 different types of towers, and with some manipulation by the end user, this might be sufficient for many types of analyses; if not, populate the Fragility sheet with as many rows as needed, and then adjust the following formula to reflect the updated Fragility sheet. Note: the rows in the Fragility table must be defined in increasing alphabetic order; the "TRUE" parameter allows Excel to pick an approximate match in case there is no exact match in the Fragility table; use this carefully! You may wish to change TRUE with FALSE for force an exact match. The "31" parameter tells Excel to return the value in the 31st column in the table array (column AE in the fragility sheet). =VLOOKUP(C2,Fragility!$A$2:$AH$20,31,TRUE). **COLUMN R is part of the model.**
- Column S. P(survive) 3 conductor. This is the chance this tower will survive if the adjacent (any adjacent) tower collapses. =VLOOKUP(C2,Fragility!$A$2:$AH$20,34,TRUE). **COLUMN S is part of the model.**
- Column T. Design Span, feet. This is the design span for this type of tower. =VLOOKUP(C2,Fragility!$A$2:$AH$20,5,TRUE). **COLUMN T is part of the model.**
- Column U. DEMAND Storm Wind (mph). This is the wind speed at the tower location from the assumed storm. In Case 1, it varies from 45 mph (SSDE at Satsop) to 67 mph

(SSDE at Aberdeen). The values 45 and 67 are highlighted to indicate that they were user entered. The intermediate values are computed within Excel based on a linear extrapolation based on the number of towers along the circuit (96); a more refined extrapolation would factor in the relative distances between the towers with the user-entered values; but this is likely a second order effect. For Case 1, these values represent the Exposure C wind speeds at the time the tower failed, assuming a liner slowdown of wind speeds from Aberdeen to Satsop. **COLUMN U is part of the model.**

- Column V, sqrt(Kzt). This is the topographic factor applied to the Exposure C wind speed for this tower. For Case 1, this is set to 1.0 (not used). **COLUMN V can be adapted to be part of the model, but is not used in Cases 1 - 8.**
- Column W, Local Demand (mph). This the wind speed that will be used on the reliability analysis, computed as `=U2*V2`. If the user wishes to enter Kzt directly in column V2, then the formula should be changed to `U2*sqrt(V2)`. **COLUMN W is part of the model.**
- Column X. Local Strength Factor. This is a factor to increase (usually decrease) the strength of the tower on each row, to take into account factors not otherwise considered in the fragility wind speeds. For example, a specific tower might be known to have been installed in weak geologic conditions, that were not factored into the design basis strength of the tower. This factor is specified in terms of Strength (like pounds, psf, etc.), so reliability calculations made using this factor must be factored by the square root of it, as all reliability wind calculations are based on comparing wind speeds. **COLUMN X can be adapted to be part of the model, but is not used in Cases 1 - 11.**
- Column Y. Local CAPACITY. This is the standard capacity (R2) times the SQRT(local strength factor) times the sqrt(design span / actual span). The results is the strength capacity of the tower in terms of externally applied wind speed. Units are mph. `=SQRT(X2)*R2*SQRT(T2/Q2)`. **COLUMN Y is part of the model.**
- Column Z. Overcapacity Parameter, on Wind. User entered. This is the conversion factor to adjust the strength capacity (in terms of wind velocity) to fragility level (in terms of mph). In Case 1, this is set as 1.12, being the best estimate of the fragility capacity of a tower designed per ASCE 74 and installed as designed. **COLUMN Z is part of the model.**
- Column AA. Local FRAGILITY, MPH. This is the strength capacity of the local tower (column Y) times the Overcapacity factors (column Z). **COLUMN AA is part of the model.**
- Column AB. Beta Wind. User entered. This is the dispersion factor for the uncertainty and randomness in the actual wind speed. For Case 1, this is entered at 0.283 for all towers. **COLUMN AB is part of the model.**
- Column AC. Beta Tower. User entered. This is the dispersion factor for the uncertainty and randomness in the actual tower capacity, in terms of wind speed. For Case 1, this is entered at 0.100 for all towers. **COLUMN AC is part of the model.**
- Column AD. Beta Total. This is computed as the SQRT (beta(tower)^2 + beta(wind)^2). For case 1, the Beta Wind and Beta Tower values are set so that Beta Total is 0.30. **COLUMN AD is part of the model.**
- Column AE. D/C Cutoff. User Entered. D represents demand (external wind speed) and C represents capacity (wind speed at which 50% of similar towers initiate collapse). For case 1, this is set at 0.50. **COLUMN AE is part of the model.**

- Column AF. D/C. This is the demand to capacity ratio for the specific tower. =W2/AA2. **COLUMN AF is part of the model.**
- Columns AG (w), AH (ax), AI (t), AJ (d), AK (p small), AL (p small semi final). These variables in these six columns are used to compute the cumulative probability function for a lognormal distribution with (D/C) and (Beta Total) as input. The calculation is continuous and accurate to six standard deviations either side from the mean. **COLUMNS AG, AH, AI, AK are part of the model.**
- Column AM. Truncated p(fail). Column AM is the same as Column AL, except that that two truncations are applied: if the actual demand to capacity ratio (D/C) is less than the User-entered D/C cut-off ratio, the probability of failure is set to 0.0; and, if the number of standard deviations is more or less then 6, the probability of failure is set either to 0.0 (<6) or 1.0 (>6.0). **COLUMN AM is part of the model.**

At the bottom of Column AM (p(fail)); ten summary statistics are provided:

- Min. The lowest p(fail) for any of the 96 towers in Case 1 (0.0)
- Max. The highest p(fail) for any of the 96 towers in Case 1 (0.0822)
- Total. The sum of p(fail) for all of the 96 towers in Case 1 (1.1081)
- Avg. The average p(fail) for all of the 96 towers in Case 1 (0.0115)
- Median. The median p(fail) for all of the 96 towers in Case 1 (0.0000)

- Number p(fail) >1%. The number of the 96 towers with p(fail) > 1% (32 towers)
- Number p(fail) >2%. The number of the 96 towers with p(fail) > 2% (25 towers)
- Number p(fail) >5%. The number of the 96 towers with p(fail) > 5% (10 towers)
- Number p(fail) >10%. The number of the 96 towers with p(fail) > 10% (0 towers)
- Number p(fail) >25%. The number of the 96 towers with p(fail) > 25% (0 towers)

The user can modify the Excel table to compute different types of summary statistics.

3.5.2 Sheet Fragility

The following describes the rows and columns in the Sheet for Case 1. These descriptions also apply to the Sheets for Cases 2 through 11.

- Row 1. This is the header row.
- Rows 2-20. These are the rows with the computations for each of the 19 different tower types
- Column A. Revised Type. This is the name assigned for each standard tower type. For BPA, the tower types are described using a coding scheme. For example, type 22L means a 230 kV tower that supports a single 3-phase circuit, without a ground wire. This column must be provided in alphabetic order, in order for the "lookup" functions in the Case 1… Case N sheets to work properly.
- Column B. Count. This column shows the number of towers of this type in the circuit to be analyzed. This data is for information only, and is not used in the analysis.
- Column C. Frag #. This value is a link to a database used for BPA. This data is for information only, and is not used in the analysis.

- Column D. Description. This is a description of the type of tower. This data is for information only, and is not used in the analysis.
- Column E. Design Horiz Span, feet. This is the horizontal span assumed in the standard design. This data is used in the analysis.
- Column F. Design Vertical Span, feet. This is the vertical span assumed in the standard design. This data is not used in the analysis.
- Column G. Design Span Factor (SF). This is the horizontal span factor assumed in the standard design. This data is used in the analysis. This is the assumed SF used for computing forces applied to conductors and ground wires to get the resultant wind force on the tower
- Column H. Design number of conductors. This is the number of conductors assumed in the design. For a single AC circuit with 1 conductor per phase, enter 3; with 2 bundled conductors per phase, enter 6.
- Column I. Design Conductor Diameter, inches. This is the diameter of the conductor assumed in the standard design.
- Column J. Design Conductor Strength, pounds. This is the nominal strength (breaking strength) of the conductor assumed in the standard design.
- Column K. Design number of ground wires. This is the number of ground wires assumed in the design. This is commonly 0 (no ground wire), 1 (1 ground wire) or 2 (2 ground wires).
- Column L. Design Ground Wire Diameter, inches. This is the diameter of the ground wire assumed in the standard design.
- Column M. Design GW Strength, pounds. This is the nominal strength (breaking strength) of the ground wire assumed in the standard design.
- Column N. Design Min Line Angle, degrees. This is the minimum horizontal angle of the conductors at the tower. For most tower types, this will be 0 degrees.
- Column O. Design Max Line Angle, degrees. This is the maximum horizontal angle of the conductors at the tower.
- Column P. Design Wind, mph. This is the wind speed for which the tower is designed to have a FS = 1.0 on its most critical primary load carrying member (or foundation). Case A (external wind).
- Column Q. Design Rime wind, mph. This is the concurrent wind speed for which the tower is designed to have a FS = 1.0 on its most critical primary load carrying member (or foundation). Case B1 (rime ice + concurrent wind).
- Column R. Design Rime Ice, inch. This is the number of radial inches of rime ice assumed on the conductor for design, with concurrent wind. Case B1.
- Column S. Design Glaze wind, mph. This is the concurrent wind speed for which the tower is designed to have a FS = 1.0 on its most critical primary load carrying member (or foundation). Case B2 (glaze ice + concurrent wind). Case B2.
- Column T. Design Glaze Ice, inch. This is the number of radial inches of glaze ice assumed on the conductor for design, with concurrent wind. Case B2.
- Columns U, V, W. These list the simplified transverse load applied to the tower at the assumed design external events (Case A, B1, B2). This information is needed to consider over-strength situations on angle towers. This information is not uses in Cases 1-11, but could be used in a refined analysis. For Cases 1-11, these are for information only.
- Columns X, Y, Z. This represents the transverse load on the tower due to conductor line tensions for the three load cases, assuming the maximum design line angle for the tower. For

Column X =H2*2*J2*0.3*SIN(RADIANS(O2/2)). For columns Y and Z, replace 0.3 with 0.4 and 0.5, respectively. For Cases 1-11, these are for information only.

- Columns AA, AB, AC. This represents the transverse load on the tower due to ground wire tensions for the three load cases, assuming the maximum design line angle for the tower. For Column AA =K2*2*M2*0.3*SIN(RADIANS(O2/2)). For columns AB and AC, replace 0.3 with 0.4 and 0.5, respectively. For Cases 1-11, these are for information only.
- Column AD. This represents the maximum transverse load on the tower, from Cases A, B1, B2.
- Columns AE, AF, AG. These columns represent the adjusted strength capacities for the tower, normalized. In this spreadsheet, we did not normalize, as the raw tower strength capacities (columns P, R, T) were already normalized. This information is used in the model.
- Column AH. This column shows the capacity of this tower to survive the failure on an adjacent tower. 1.00 means the tower can always resist the failure of an adjacent tower. This information is not used in file Satsop-Aberdeen 4.xls, but is used in the cascading (zippering) model described in Section 6 of this report.

3.6 Reliability Model Results and Discussions (Level 1)

Sections 3.6 and 3.7 describe 11 different reliability analyses (Cases 1 to 11) performed for the Satsop-Aberdeen #2 transmission line. Cases 1 through 8 can be considered a "Level 1" analysis, as these take the least effort. Cases 9 through 11 require additional input variables, and can be considered a "Level 2" analysis.

Case 1 begins with the simplest assumptions, and each subsequent case examines the variability and refinement in results as certain assumptions are relaxed / refined. Table 3-3 summarizes key results for all Cases 1 through 8.

Case	Level	Wind Speed MPH (U, W)	Beta Wind (AB)	Beta Tower (AC)	D/C Cutoff (AE)	Tower Failures (initiations)	Towers with p(fail) > 10%	Towers with p(fail) > 25%	Tower 14179 p(fail)
1	1	67 – 45	0.28	0.1	0.50	1.11	0	0	0.065
2	1	70 – 50	0.28	0.1	0.50	1.86	2	0	0.089
3	1	67 – 45	0.28	0.1	0.60	0.66	0	0	0.065
4	1	67 – 45	0.30	0.26	0.60	1.33	11	0	0.128
5	1	67 – 45	0.30	0.26	0.55	1.95	12	0	0.128
6	1	56 – 45	0.30	0.26	0.50	0.89	0	0	0.062
7	1	56 – 45	0.40	0.30	0.50	1.62	14	0	0.110
8	1	56 – 45	0.40	0.50	0.50	2.56	16	0	0.169

Table 3-3. Parameter Study, Level 1

Case 1. Wind speed at Aberdeen was 67 mph (west end nearest the ocean). Wind speed at Satsop was 45 mph (east end, inland). D/C truncation = 0.5. Beta (wind) = 0.28. Beta (tower) = 0.10. Beta (total) = 0.30. Topographic effects: ignored. Cascade failures: ignored.

The Case 1 model predicts 1.11 tower failures (initiation). Since there was actually 1 failure, the question is, is the model really this good?

Figure 3-11 shows the computed p(fail) forecast for each tower (Case 1 column AM) along the alignment.

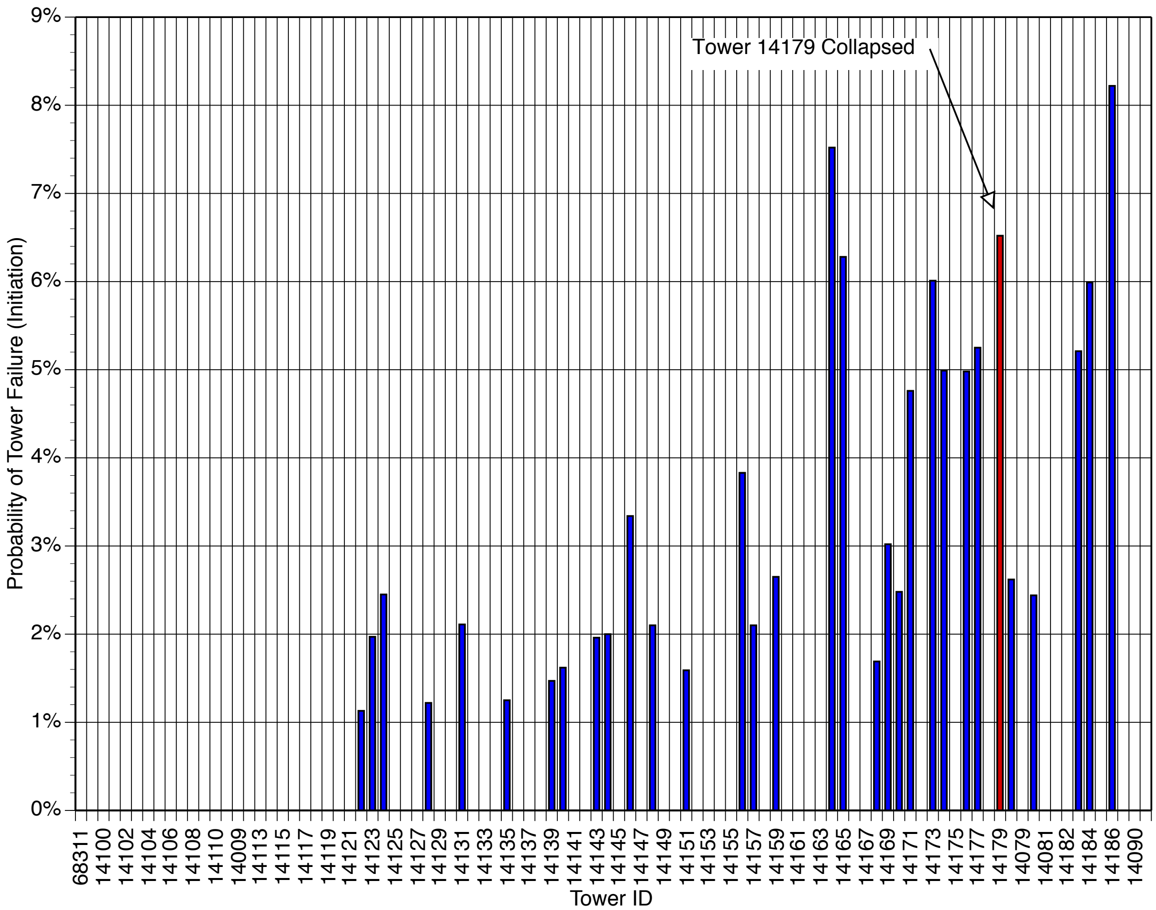

Figure 3-11. Case 1. Satsop-Aberdeen Tower Failure Probabilities

While the Case 1 analysis does highlight that Tower ID 14179 has a modestly high chance of failure (6.5%, highlighted in red in Figure 3-11), it also highlights 32 total towers with p(failure) > 1%. If one were to just rely on the Case 1 analysis, and if one were to implement a tower upgrade effort, it would appear prudent to upgrade towers 14123, 14124,… 14186 (all towers with p(fail) over about 2%, or a total of 25 towers. Even if we selected a higher cut-off rate (say p(failure) > 5%), there would still be 10 towers to be upgraded.

Case 2. We assumed in Case 1 that the wind speed field varied linearly from 67 mph (Aberdeen substation along the coast) to 45 mph (Satsop substation, inland) (Column U). We actually do not know the true wind speeds during this storm. We performed a Case 2 analysis as follows (also, see Table 4-1) by increasing the wind speed to 70 mph (west) to 50 mph (east). For Case 2, the number of failures increases from 1.11 to 1.86 (68% increase) for an average 4 mph speed increase (7%

increase on speed, 15% increase on force applied to the towers). This shows that the predicted number of tower failures is quite sensitive to a modest increase in wind speed.

Case 3. Both Cases 1 and 2 assume that the lognormal fragility distribution for tower capacity is truncated at D/C = 0.50. In other words, there is no chance of tower failure if the applied demand (wind speed) is less than 50% of original fragility capacity (wind speed). This low value for truncation is disturbingly low, in that full scale test data of actual towers (sample size about 100 towers) has shown that the actual lower bound strength at perhaps 90% of ultimate (or perhaps 80% at the extreme).

Given this issue, we ran Case 3, which is the same as Case 1 but with D/C cutoff set at 0.60. In this case, the model show 0.66 tower failure initiations. This suggests that the Case 1, 2 or 3 models are missing some fundamental factor.

Cases 4, 5. We then allowed beta to increase to reflect uncertainty. The model is set up with two betas (one for wind speed, one for tower capacity). From a computational point of view, beta (total) is most important, computed as the square root of the sum of the squares of the individual betas. For Case 4, we begin with Case 3, and then increased beta total to 0.40. The results show 1.33 tower failures (Case 4) or 1.95 failures (Case 5 with truncation at 0.55).

In examining the actual mode of failure for these towers, we observed that the foundations pulled up due to overturning moment, almost like a shaft. However, review of the original foundation design (grillage footings) assumed that the uplift forces on the tower leg would be resisted by a cone-shaped soil wedge. Further review showed that the top layer of soil in these hills was formed by volcanic ash. Normally, this ash is weakly cemented, and provides fairly good resistance, as was assumed in original design. However, it was observed in the actual storms that the top soil layers had become saturated, and the soil lost most of its strength, leading to much lower capacity than originally assumed. This condition was unknown at the time of the original design and installation. Given this knowledge, we believe it reasonably to assume that the true strength of at least some of these towers along this alignment is controlled based on the local foundation soil conditions, which remain uncertain as we have not performed subsurface investigations at each tower location.

Case 6. Clearly, one of the primary variables in predicting the damage is the actual wind speeds. The nearest weather station is at the Hoquiam airport. The runway of this airport can be seen on the upper left side of Figure 3-12, next to the water). The common abbreviation for the weather station at the Aberdeen, Washington airport is KHQM (K = US Station, HQM = airport).

Figure 3-12. Aerial Photo, Aberdeen, Washington

The following data is available from this weather station (www.wunderground.com) for the day of the actual storm that damaged the tower:

- Min 42°F Max 50°F. Precipitation: 0.95 inches. Max wind speed 50 mph. Max Gust speed 56 mph. Note: the duration of max wind and gust wind speeds is not defined, but presumably it is based on the instrument with intent to be useful for pilots. It is assumed that max gust speed represents a 3-second wind, but we have not checked the actual instrument to verify this.

The storm of February 23 - 24, 1999 that damaged the transmission tower was a winter storm event. Figures 3-13 and 3-14 provide charts for wind speeds and directions at the airport. The charts suggest the storm peaked between 8 pm and 11 pm Feb 23, but no data is now available for the time from 11 pm (pacific standard time) Feb 23, 1999 to 6 am Feb 24, 1999. The actual tower collapse was reported at 11:53 pm local time. It is thought that the weather instrument failed (lost power or was damaged) after 11 pm, or the airport was closed and the instrument turned off sometime after 11 pm. The direction of the wind was from the south.

Examining the wind data, if we use the maximum gust speed of 56 mph as recorded at KHQM, we would predict 0.89 tower failures for the entire circuit (Case 6) with beta total = 0.4 and D/C truncation at 0.50.

Cases 7, 8. For Cases 7 and 8, we increase beta to 0.5 (Case 7) or 0.64 (Case 8). It is hard to fathom if beta can be much larger than 0.6, and it is hard to fathom that the truncation cut-off can be lower than 0.55. Therefore, we reject the hypothesis that the maximum actual gust speed along the alignment was under 56 mph.

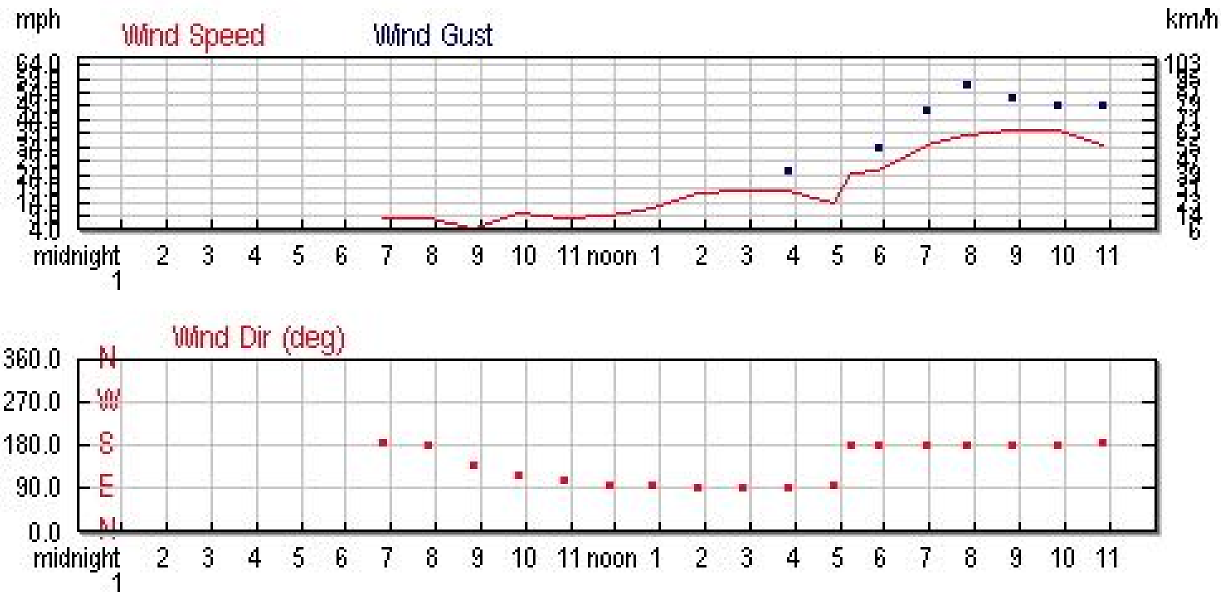

Figure 3-13. Weather Station Data, February 23, 1999

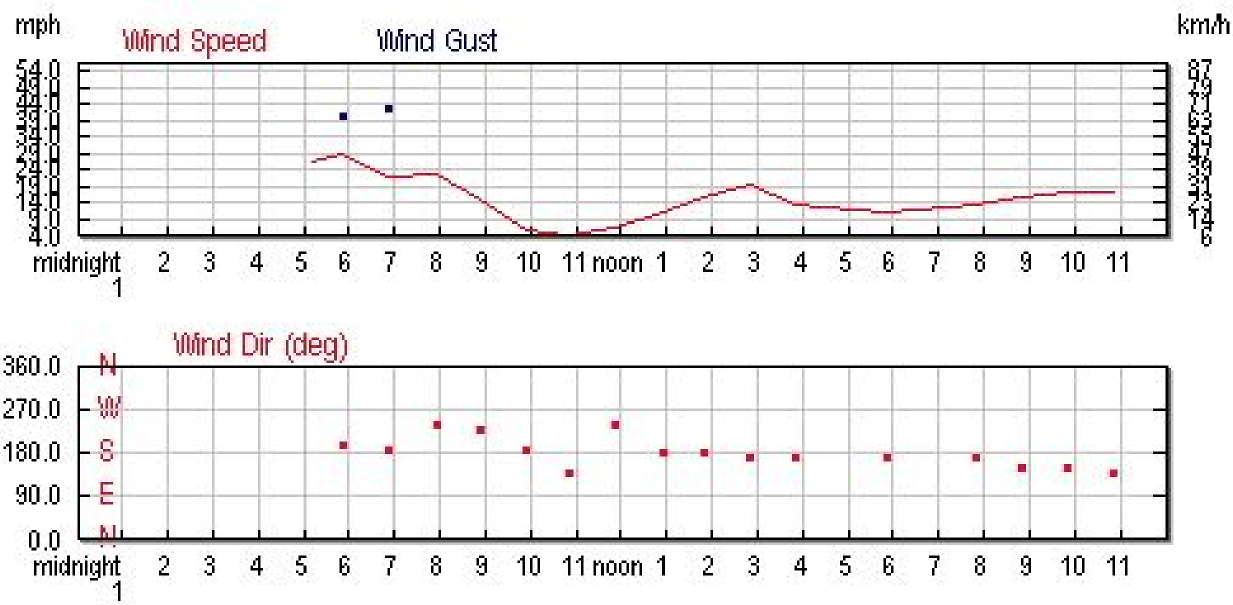

Figure 3-14. Weather Station Data, February 24, 1999

The KOLM station was for Olympia, WA (east of the Satsop substation), reporting 44 mph max gust speed on Feb 23, 1999. Wind direction from the south.

From the available weather station data, it is reasonable to assumed that there was about a 10 to 15 mph gradient in the wind speed from Aberdeen to Olympia.

Summary, Cases 1 – 8.

Cases 1 through 8 represent a "Level 1" type of analyses. The basic information about tower locations and types are input to the model, as well as Exposure C type information about wind speeds that occurred. Eight cases were examined to see the sensitivity of variation in certain model variables. The overall tower failure initiations predictions (Table 3-3, from 0.66 to 2.56) are within the bounds of reason as 1 tower (number 14179) actually did fail.

In order for the Level 1 analysis to provide reasonably-predictive results, the D/C truncation needs to be set quite low, and the beta parameters quite high, reflecting that there is potentially a lot of uncertainty in this Level 1 model.

Key findings from Cases 1 to 8:

- A comparison of the actual span versus the design span lengths appears to be a good first order indicator as to which tower(s) are potentially the weakest and potentially overloaded.
- For these BPA towers, cascading effects are unlikely, and zippering did not occur in the actual storm. Section 6 of this report will examine cases when cascading effects are important.

In summary, the Level 1 model can be used to predict damage to this transmission line. In order for the Level 1 analysis to provide rational results, the D/C truncation needs to be set quite low, and the beta parameters quite high, reflecting that there is potentially a lot of uncertainty in the model. Key findings:

- A comparison of the actual span versus the design span lengths is the first order indicator as to which tower(s) are potentially the weakest and potentially overloaded.

- For these BPA towers, cascading effects are unlikely, and do not contribute much to the overall risk.

3.7 Reliability Model Results and Discussions (Level 2)

With the intent to develop a more refined Level 2 model, by examining the results from Cases 1 through 8, we observe the following:

- Tower 18/2 (ObjectID 14179 and Tower 18/3 (ObjectID 14180) are located either side of a gully, and were loaded by southwest-to-northeast winds during the storm. (See Figures 3-5, 3-6, 3-7)
- Tower 18/2 is a lighter structure (22L) than Tower 18/3 (2H1), by design.
- The actual tributary span for 18/2 was 1,255 feet, or 55 feet longer than its nominal design span of 1,200 feet.
- The actual tributary span for 18/3 was 1,376 feet, or 74 feet shorter than its nominal design span of 1,450 feet.

- The nominal wind strength capacity of 18/2 is 92 mph * sqrt(1200/1255) = 90 mph (FS = 1.0).
- The nominal wind strength capacity of 18/3 is 100 mph * sqrt(1450/1376) = 102.7 mph (FS = 1.0).

Based on this analysis, we would expect that 18/2 is 30% weaker than 18/3 (on lateral force) or 14% weaker (on wind speed) for the actual installation. Therefore, it is not surprising that 18/2 was the tower that failed.

Since there were no local anemometers at towers 18/2 or 18/3 to measure the actual wind speed during this storm, the following observations are made:

- What was the actual wind speed across the Bear Gulch gully during the storm? If there were no construction defects / age effects on the towers, then we can surmise that the true wind speed (at elevation above ground of Z=10m) was at least 90 mph, with the best estimated wind gust speed about 100.8 mph (assuming 50% tower failure at (FS=1) * 1.12, based on tests).

- The design wind speed (3-s at Z=10m) at this location, using the ASCE 7-1995 or 7-2005 maps, is 85 mph (50 year return period); using the ASCE 7-2010 map is 108 mph (700 year return period). All the ASCE maps denote the coastline in this area as a "special wind region", requiring local investigation.

- Using the Scenario storm wind map (Figure A-7) for the Pacific Northwest, wind speeds at the coast tower site are not to exceed about 100 mph (5-sec Z=10m), and perhaps just 95 mph or so at this site.

By examining the local topographic maps (Figures 3-5 and 3-6), we see that the Bear Gulch gully is aligned ideally to promote funneling of the wind from the south or southwest into the gully. This type of wind-speed up process is not captured in any of the country-wide or regional maps in Appendix A. The challenge is performing the reliability analyses is as follows:

- If we ignore the local topographic features in hilly terrain (like Bear Gulch gully, and like what was done in Cases 1 through 8), then we are left to use the regional or country-wide wind-loading maps. Models that forecast tower damage using these types of Exposure C maps that ignore topographic effects will likely produce unreliable results.

So, to perform a Level 2 analysis, we wish to include the effects of local topographic relief. SEAW (2004) provides a topographic model that can be applied tower by tower. This topographic model uses actual hill shape and other variables to develop Kzt, a topographic wind-speed-up factor. Application of the SEAW model would take a few minutes of engineering time for each tower, to consider the actual topography. If the complete topography for an area is available in digital fashion, a computational model could be developed to do this calculation in a second or so; but this is not done for this report. Instead, we use the average slope for the area near a tower as a proxy for the true computation of Kzt. The proxy formulation adopted to rapidly compute the topographic wind speed-up (Kzt) for each tower is as follows:

- If local slope is more than 40%, assume Kzt = 2.0
- If local slope is between 20 to 40 %, assume Kzt = 1.25
- If local slope is under 20%, assume Kzt = 1.00 (no effect)

This simple set of rules is easily adopted in the Excel spreadsheet model, although it is not quite the same as the SEAW (2004) model. However, it provides wind-speed-up values that appear rational and within the range of likelihood.

Table 3-4 lists the summary results for Cases 9, 10 and 11.

Case 9. For Case 9, we begin with the Case 8 factors, and include the Kzt term. For Case 9, the number of tower failures increases to 8.69 (too many), and the chance of failure for tower 18/2 is 55%.

Case 10. For Case 10, we begin with Case 9, but reduce the beta terms to the original assumptions for Case 1. For Case 10, the number of tower failures is 4.52 (still too many), and the chance of failure for tower 18/2 is 60%.

Case 11. For Case 11, we begin with Case 10, and set the truncation to 0.75. For Case 11, the number of tower failures is 3.23 (high, but possibly within reason), and the chance of failure for tower 18/2 is 60%.

Case	Kzt (V)	Exposure C Wind Speed MPH (U)	Local Wind Speed MPH (W)	Beta Wind (AB)	Beta Tower (AC)	D/C Cutoff (AE)	Tower Failures (initiations)	Towers with p(fail) > 10%	Towers with p(fail) > 25%	Tower 14179 p(fail)
9	Yes	56 – 45	109 – 45	0.40	0.50	0.50	8.69	32	14	.55
10	Yes	56 – 45	109 – 45	0.28	0.10	0.50	4.52	11	7	.60
11	Yes	56 – 45	109 – 45	0.28	0.10	0.75	3.23	7	7	.60

Table 3-4. Parameter Study (Level 2)

Given these findings from Cases 9 – 11, we are able to get the p(fail) for the actual tower 18/2 that failed up to 55% to 60%. This is a high enough percentage to be a good indicator that there is something particularly weak for this tower 18/2, as well as to create a rational upgrade program to retrofit the weakest 7 towers along the alignment.

The findings from Cases 1 through 11 also show that the reliability model must have the following features in order to be realistic:

- Actual span lengths need to be accurate.
- Design basis for each actual tower must be quantified.
- Local topographic effects for wind speed-up are important. If these are not known (Level 1), then reasonable results can be predicted, but they will be more uncertain.
- The D/C truncation is an important parameter. While past test data suggest a truncation of perhaps 90%, actual in-situ construction (local geologic conditions) can result in much lower capacity than assumed in the design, and thus invalidate the D/C truncation observed from controlled test.

- The beta values for the tower can wind speeds are important, but at least for this example, the results are not overly sensitive to the assumptions.

One of the important findings is that some of the towers may have been constructed with foundations in soils that are considerably weaker than originally assumed during original design and construction. For this particular circuit, some towers have foundations in a volcanic-ash-deposited material that becomes much weaker when saturated than when the material is dry. This means that the circuit is more reliable for a summer storm (dry conditions) than a winter storm (ground-saturated conditions).

To account for such local conditions, a column (X) is included in the model to allow the user to selectively enter a strength increase or decrease factor for each tower (in the Case 1…11 sheets), rather then relying on the generic strengths for all towers of a specific type (from the Fragility sheet). This type of tower-specific modification might be important in many cases. If the user wishes to use this factor, then enter a factor that corresponds to the strength (in pounds, psf, etc.) in column X; this will be factored into the tower-specific fragility via a square root function (Column Y).

3.8 Spatial Variation of Wind Speed - Hurricanes

In performing the analyses in this example, the spatial variation of the wind speed over the geographic extent of the transmission circuit was considered. This was done by varying the wind speed from about 70 mph (near the coast) to about 50 mph (inland).

For hurricanes along the Gulf and Atlantic coasts, this spatial variation can be approximated for each storm by assuming a storm track for a specific storm, and then estimating the width of the low pressure zone, and the corresponding wind field. This type of calculation can be performed, but a highly simplified approach, described below, might be reasonable.

- For much of the Gulf and Atlantic coasts, it would not be unreasonable to assume for the 2,500 year event a direct hit at a specific location (perhaps the worst location on the circuit).

- For the 100 year event, assume a hurricane that makes land about 15 miles distant from the most vulnerable point along the transmission line.

- Depending on the milepost along the coast (Figure A-8), for the 2,500 year event choose category 5 hurricane (milepost below 1500), Category 4 (milepost 1500 to 2000); Category 3 (milepost from 2000 to 2250), or Category 2 (milepost 2250 to the Canadian border, and all of Canada's Atlantic coast). In certain types of analyses, it might be useful to examine a 10,000 year hurricane event; an approximation of such an event could be done to select a wind speed for a 2,500-year event at a not-to-exceed level of 25%.

- For cases where the transmission line alignment is perpendicular to the coast, this approach will be highly sensitive to the actual assumption of landfall. For transmission lines that run long distances and are nearly parallel to the coast, this approach should not be too sensitive to the landfall location.

4.0 SAMPLE ANALYSIS 2: WIND EVENT

In Section 4, we present a reliability analysis for a 500 kV transmission line that failed in a high wind event on November 16, 2010. This line is owned by the Bonneville Power Administration (BPA). We use the model developed in Section 3 of this report. To differentiate between the models, we call the Cases 101, 102… for this 500 kV circuit.

4.1 Location of Circuit and Collapsed Tower

Figure 4-1 shows a regional map of Idaho, highlighting the location of the Hatwai – Dworshak 500 kV circuit. Structure 10/2 collapsed during a high wind event on November 16, 2010.

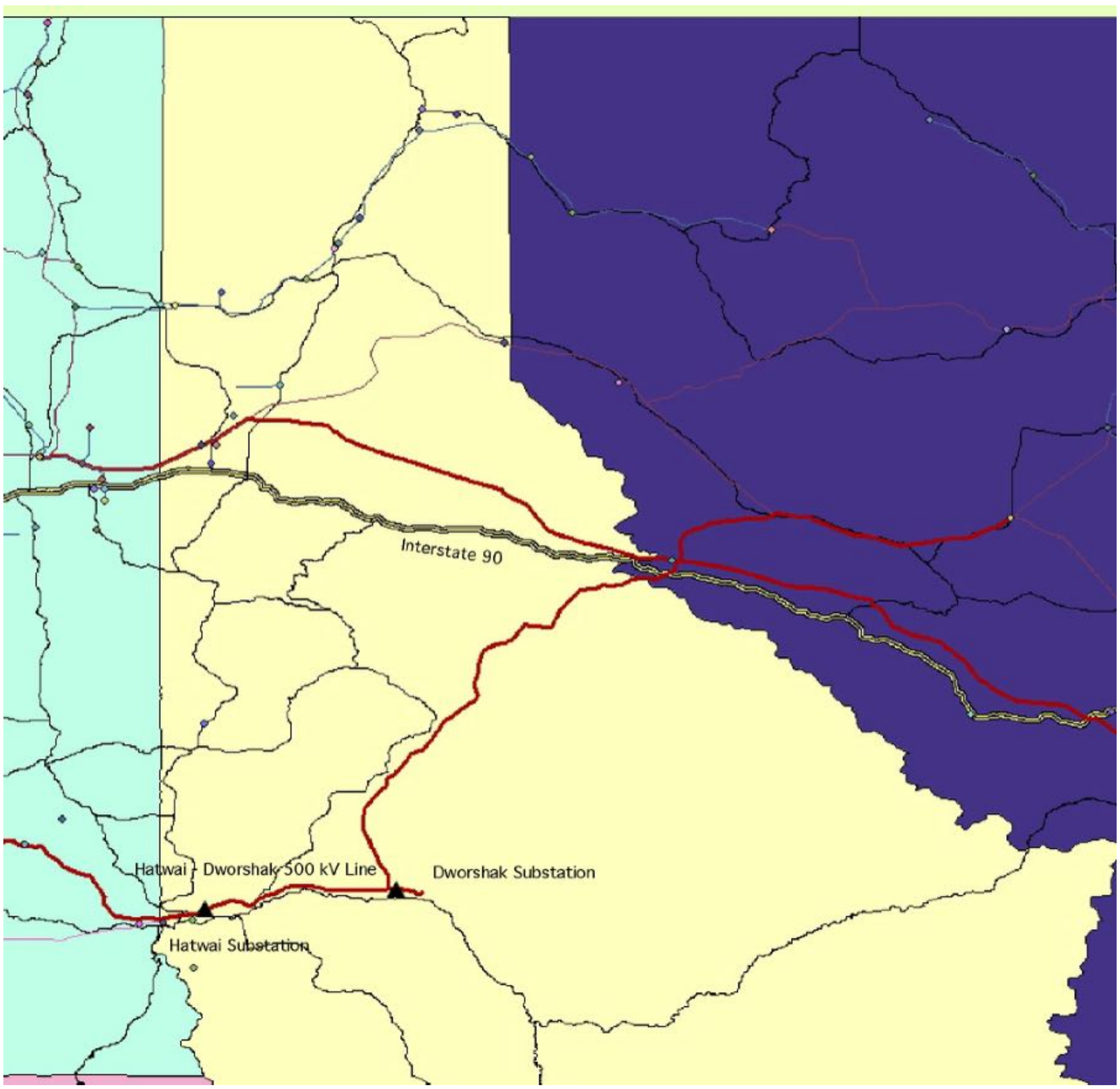

Figure 4-1. Hatwai – Dworshak 500 kV Line

Figure 4-2 highlights the location of each tower (large open circle) along the Hatwai-Dworshak line.

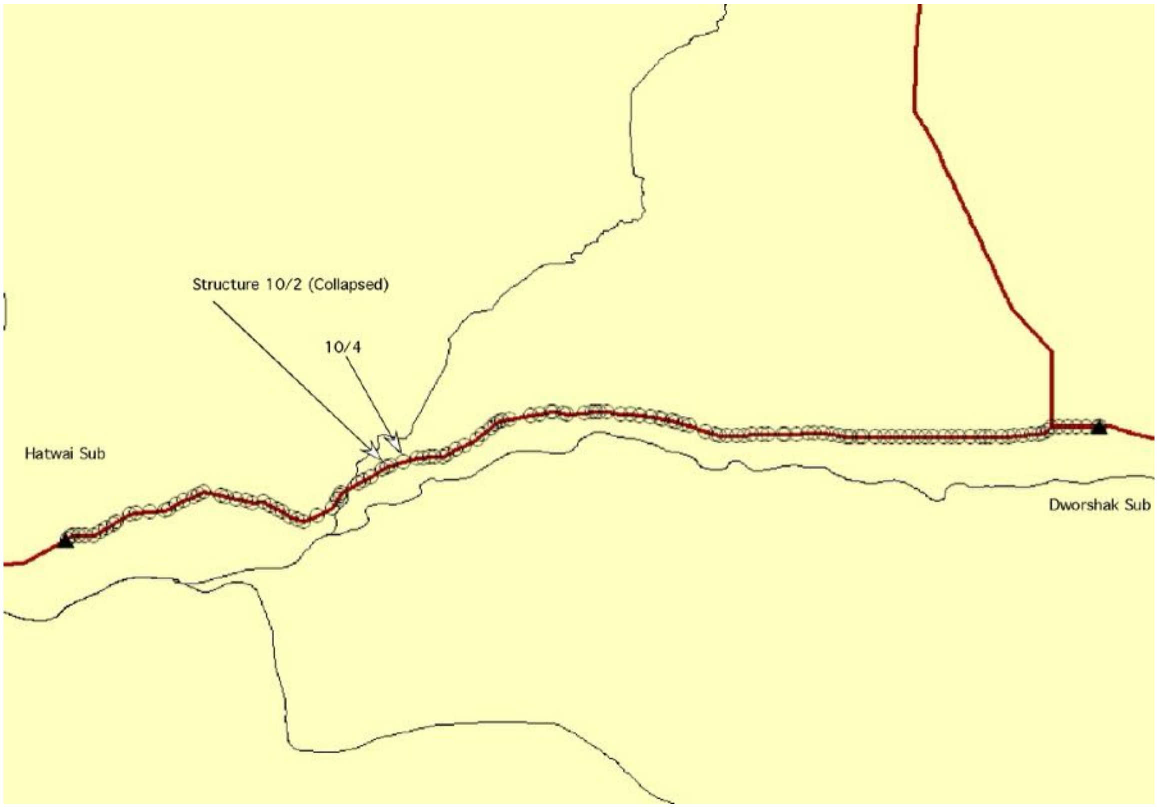

Figure 4-2. Location of Structure 10/2 on the Hatwai – Dworshak 500 kV Line

Figure 4-3 shows an aerial view of the circuit in the vicinity of the collapse. The four towers seen in this photo are 9/3 (lower left), 10/1 (middle), 10/2 (tower that collapsed near the summit), 10/3 (top right). The tower that collapsed is structure 10/2, objectID 53462. As can be seen, the terrain is largely bare near the summit (no trees), suggesting a smooth surface that will aid in increasing wind speed.

Figure 4-3. Aerial Photo, Structures 9/3, 10/1, 10/2, 10/3

Figure 4-4 shows the topographic relief for the local hill, named Myrtle. The "X" symbol is the location of tower 10/2 that collapsed. As can be seen, tower 10/2 is located nearly atop the hill.

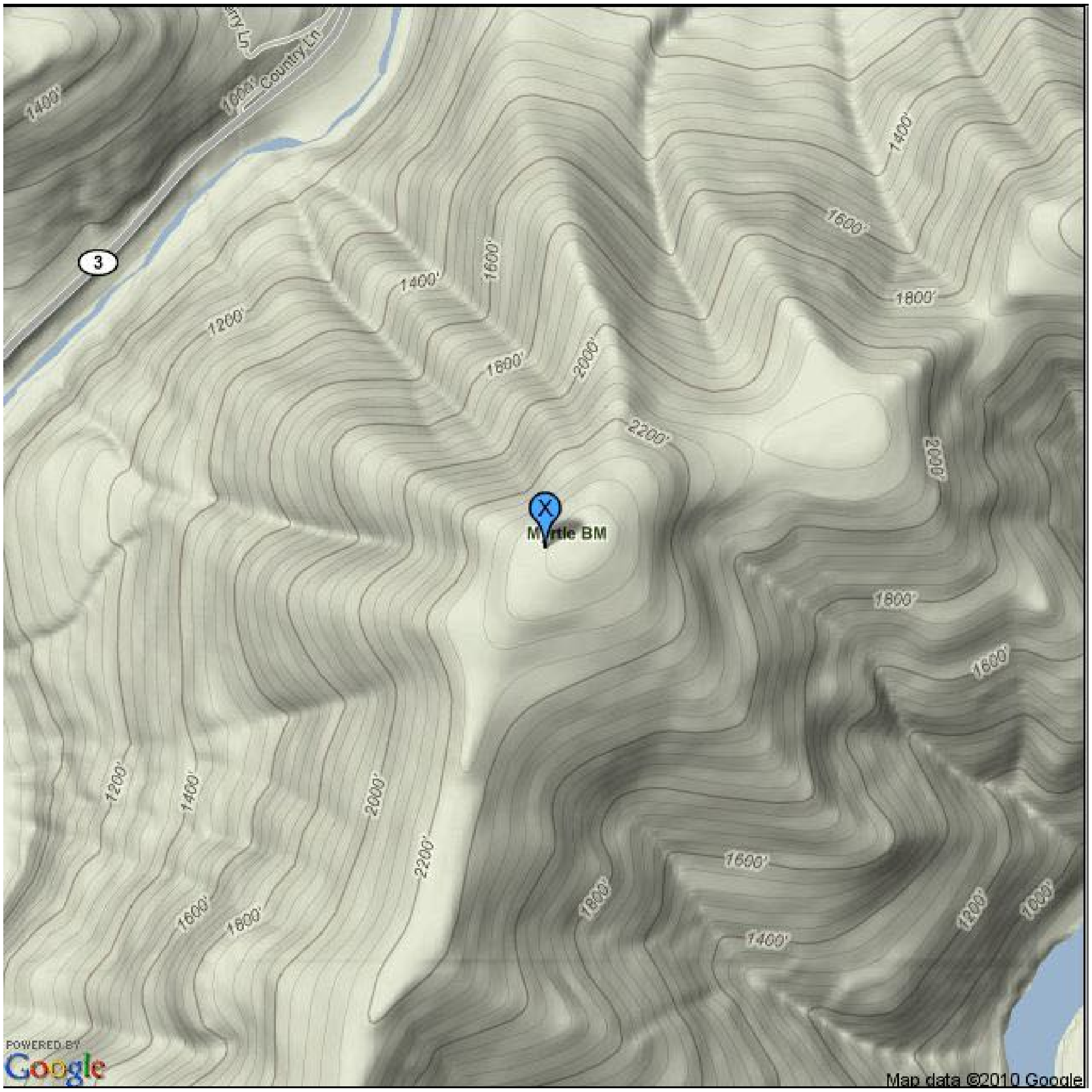

Figure 4-4. Topographic Map, Structure 10/2 (Collapsed), Type 28MV1, Elevation 2,362 feet

Figures 4-5, 4-6 and 4-7 show the locations of the adjacent towers, 10/1 (28B2, heavy tangent) and 10/3 (28DW, dead end) and 10/4. None of these towers collapsed.

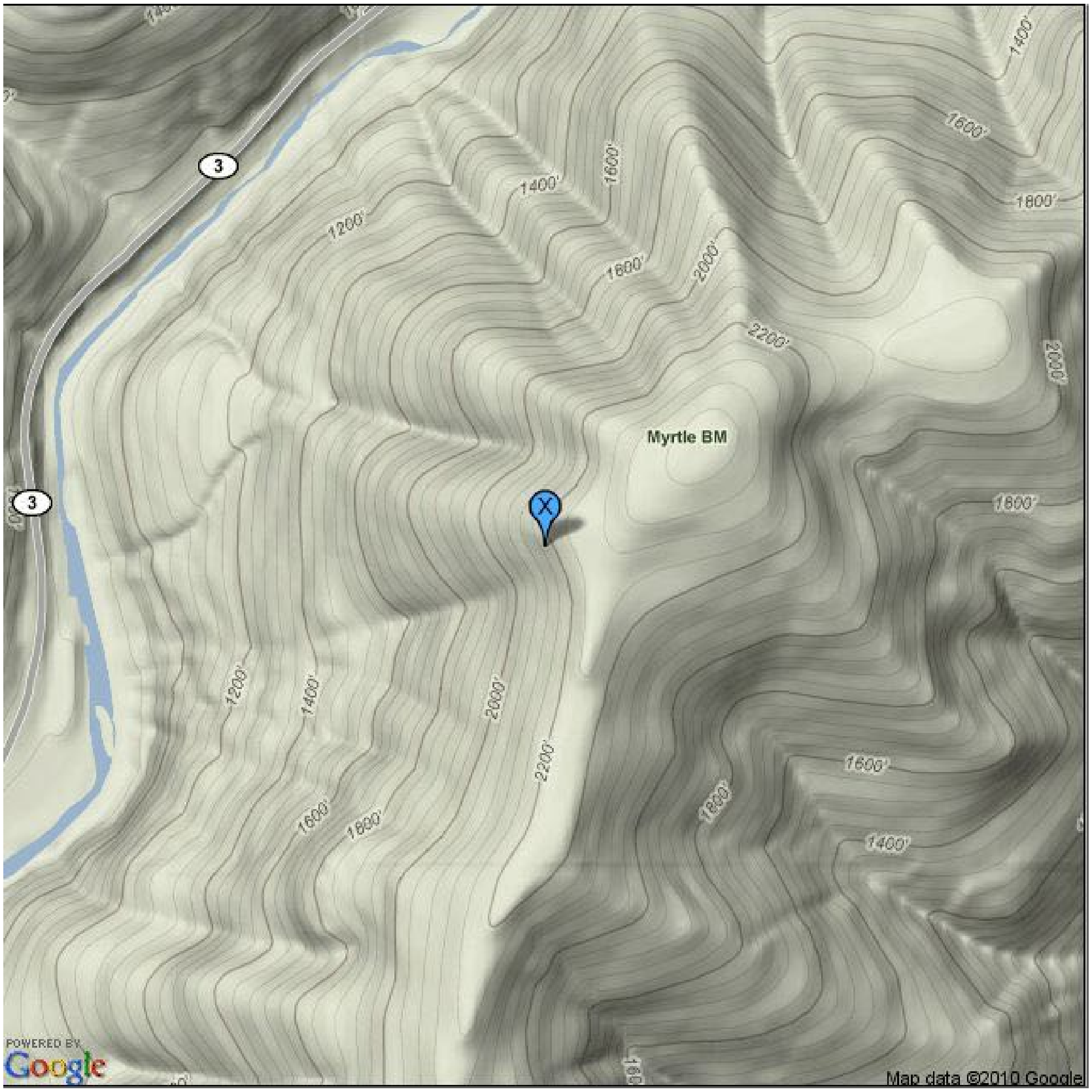

Figure 4-5. Topographic Map, Structure 10/1, Type 28B2, Elevation 2203 Feet

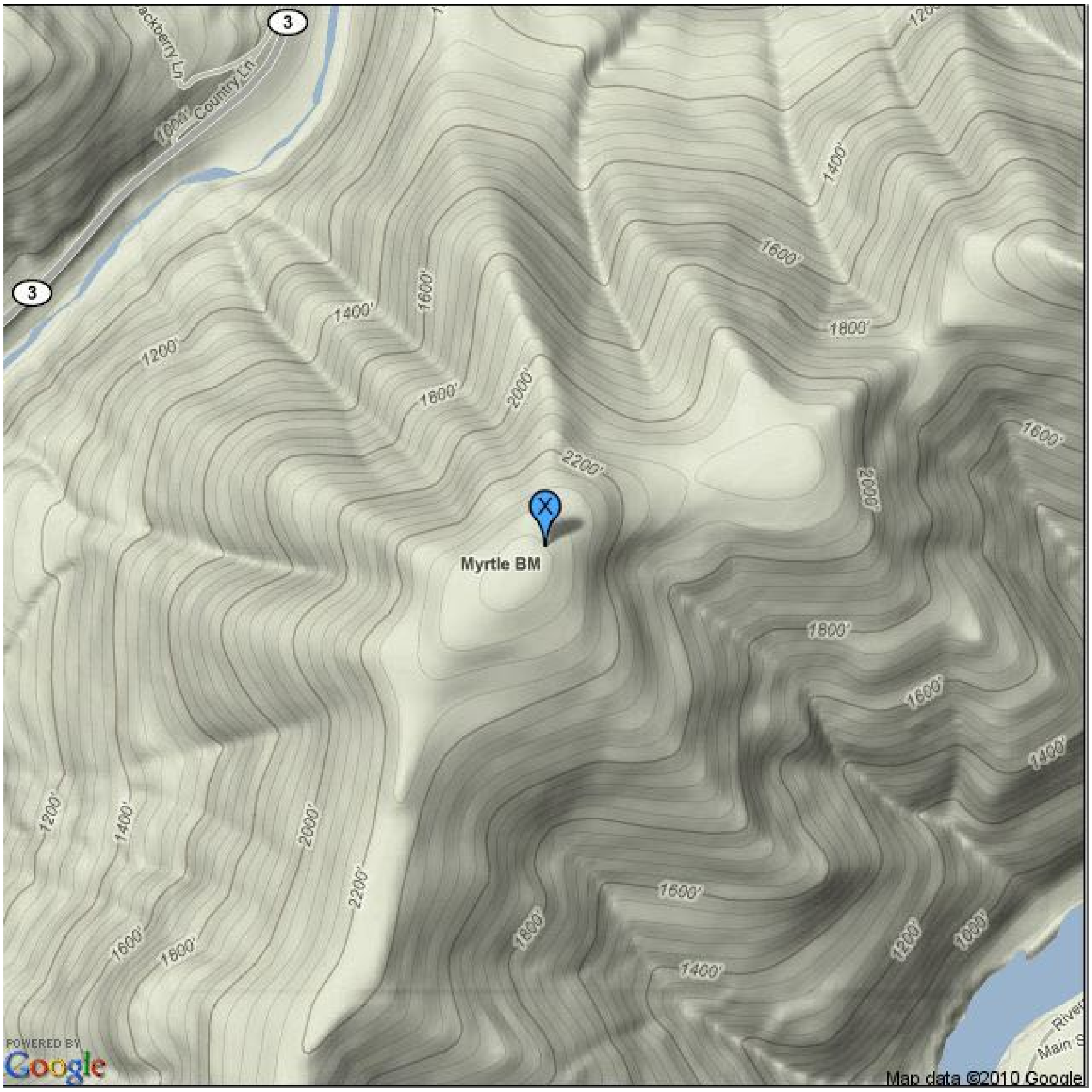

Figure 4-6. Topographic Map, Structure 10/3, Type 28DW, Elevation 2345 Feet

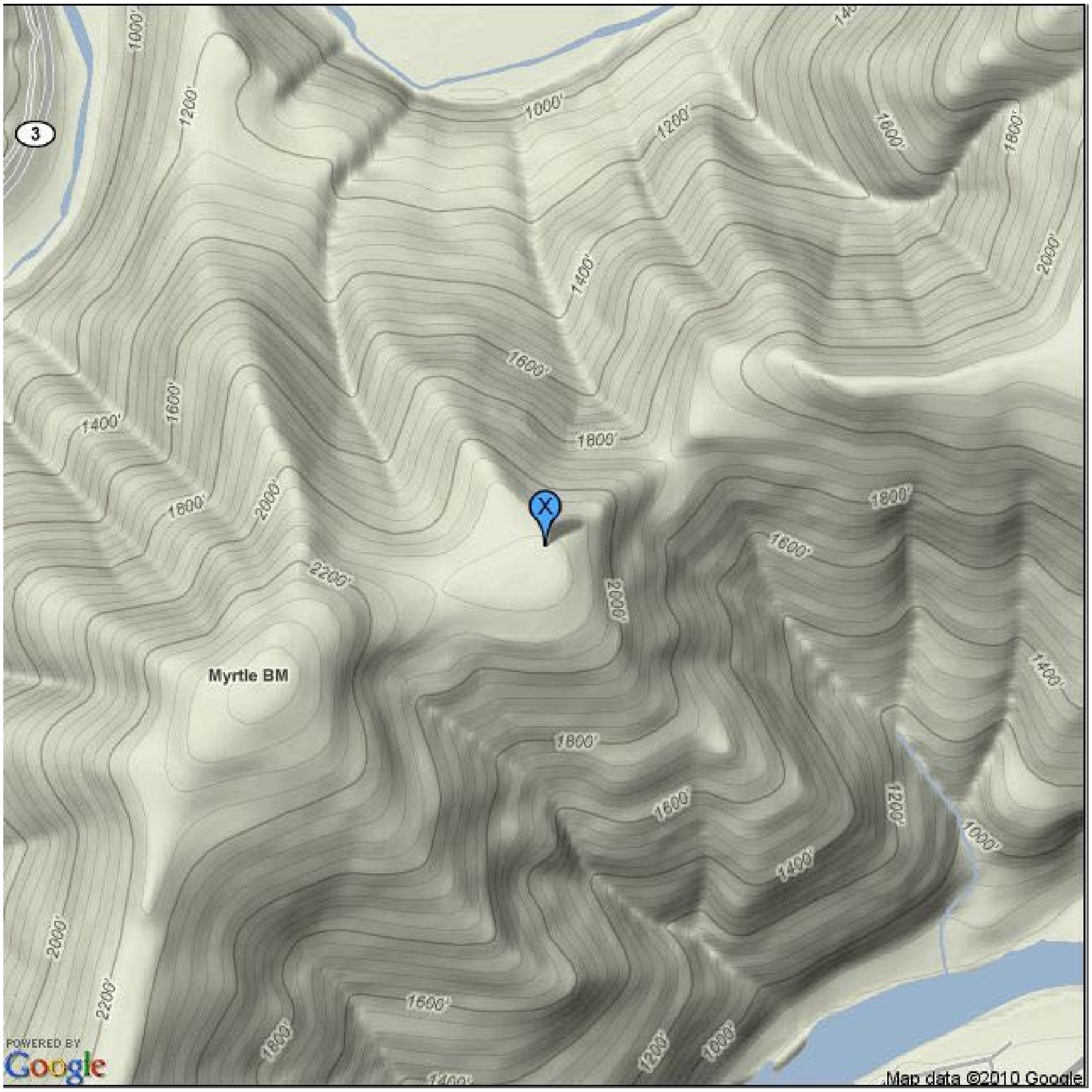

Figure 4-7. Topographic Map, Structure 10/4, Tower 53464 (Non Collapsed Tower) Elevation 2127 Feet

4.2 Circuit Information

The Hatwai-Dworshak No. 1 line is a single line circuit, operating at 500 kV. The line is located entirely in Idaho. This circuit includes 131 towers. The horizontal length of the circuit is 150,371 feet (28.5 miles).

The conductor type on this line is Bunting (diameter 1.302 inches), bundled with 3 conductors per phase, 9 conductors total. The ground wire on this line is a 7 No. 8 Alumoweld, diameter 0.39 inches.

According to information obtained from BPA, this circuit was constructed in the 1960s.

Table 4-1 lists all the types of towers and average / maximum horizontal spans along the Hatwai-Dworshak line.

Tower Type	Count	Max Horizontal Span Feet	Average Horizontal Span Feet
18DW	2	960	935
18GW	2	1620	1587
18M1	2	1148	1085
28AV1	3	1283	1260
28AV2	1	508	508
28AV4	3	1352	1232
28B1	8	1896	1637
28B2	7	2283	1492
28DW	28	2084	1289
28MV1	50	1153	948
38M1	14	1360	1200
38M2	7	1355	1165
8B1	2	1390	1340
SSDE	2	286	230

Table 4-1. HTWA-DWOK-1 Circuit Tower Types

The most common type of tower on this circuit is the 28MV1 (50 towers). The tower that collapsed was a 28MV1. Tower 28MV1 is a single circuit 500 kV tower, light tangent suspension, designed to accommodate a ground wire. The 28MV1 is a common type of tower in the BPA system, with 1,595 such towers throughout the system as of 2005.

The following information is available for the structure 10/2 that collapsed.

- Slope at the tower legs: 13.83% (slope is measured in percent, such that 100% = 45 degrees; 13.83% = 7.9 degrees). This slope is computed using one 10m x 10m horizontal grid, directly beneath the tower legs.

- Slope averaged over 150 x 150 meters near the tower (225 planes): mean: 17.28%, maximum: 57.15%.

- Estimated Vs30 = 760 m/sec (shear wave speed in top 30 m of soil). This site would be classified as a "rock" site, possibly with a thin layer (under 30 feet thick) of soil over the underlying rock. No site specific geologic data is available.

The horizontal spans for the collapsed tower 10/2 are 468 feet (ahead), and 974 feet (behind). The tributary horizontal span for this tower is 721 feet. Figures 4-14 and 4-15 show actual spans and angles along the entire circuit.

The nominal wind speed (strength capacity) for a 28MV1 tower is 127 mph applied to the conductors. This is the wind speed that would nominally bring the foundation capacity due to the applied overturning moment to the ultimate limit level. This assumes a design span of 1,200 feet, a span factor of 0.7.

The strength capacity for similar types of towers are as follows:

28M1. 1,200 foot design span. 115 mph. (2 conductor chukar, diameter 1.6 inches)

28M2. 1,200 foot design span. 115 mph.

By examining Table 4-1, we see that the maximum span for the 50 28MV1 towers along this circuit is 1,153 feet, with an average of 948 feet.

Figure 4-8 shows a photo of a 28M1 tower. Figure 4-9 shows a ATADS model of a 28M1 tower. The 28M1 model shown in Figure 4-9 slightly differs from the 28MV1 that failed in that the 28M1 has two extensions for ground wires. The variations in the 28MV1 tower as compared to the 28M1 tower are thought to be minor, so we make the simplifying assumption that the tower capacity of the 28MV1 is the same as the 28M1.

Figure 4-8. Photo of Generic 28M1 Tower (not the tower that collapsed)

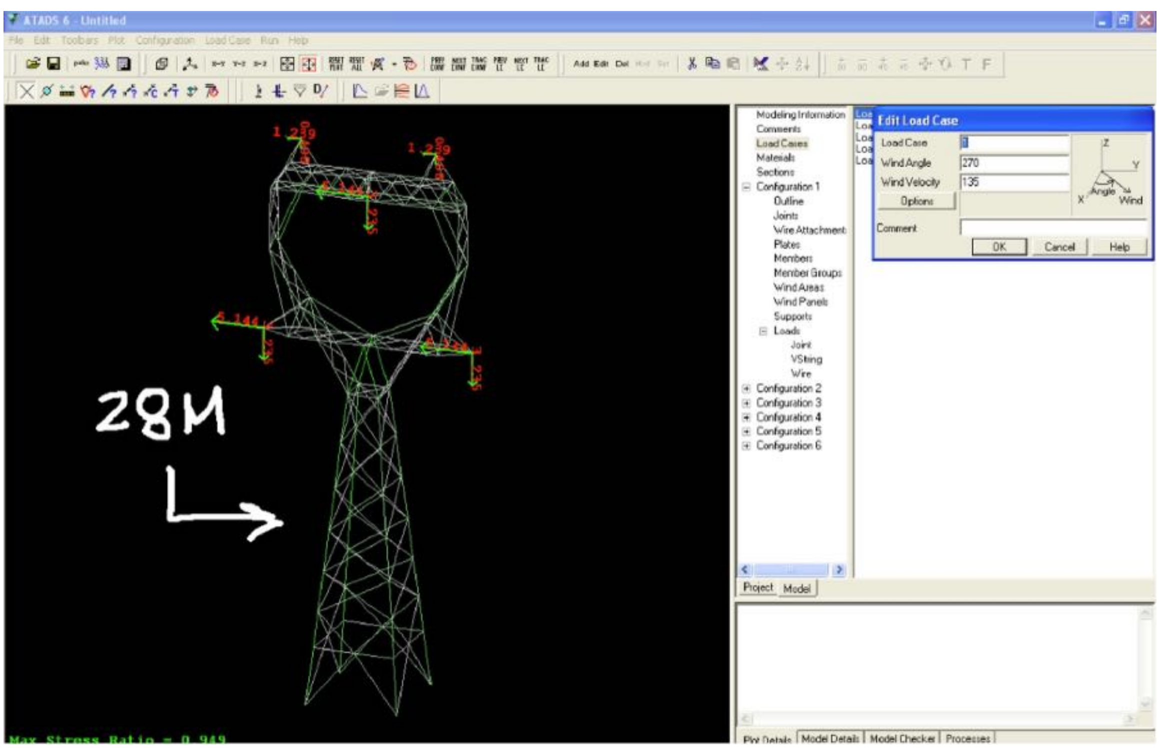

Figure 4-9. ATADS Model of 28M1 Tower

4.3 Weather Conditions, November 16-17 2010

Figure 4-10 shows a regional map, highlighting the location of tower 10/2 that collapsed (yellow pin).

Figure 4-10. Regional Map

There are two airports with available weather data in the vicinity of the failed tower. These airports are highlighted in Figure 4-11 (blue airport symbols within a circle) in relation to the collapsed tower (yellow pushpin within an oval).

- Pullman-Moscow, located about 39 km northwest of Tower 10/2.
- Lewiston, located about 26 km southwest of Tower 10/2

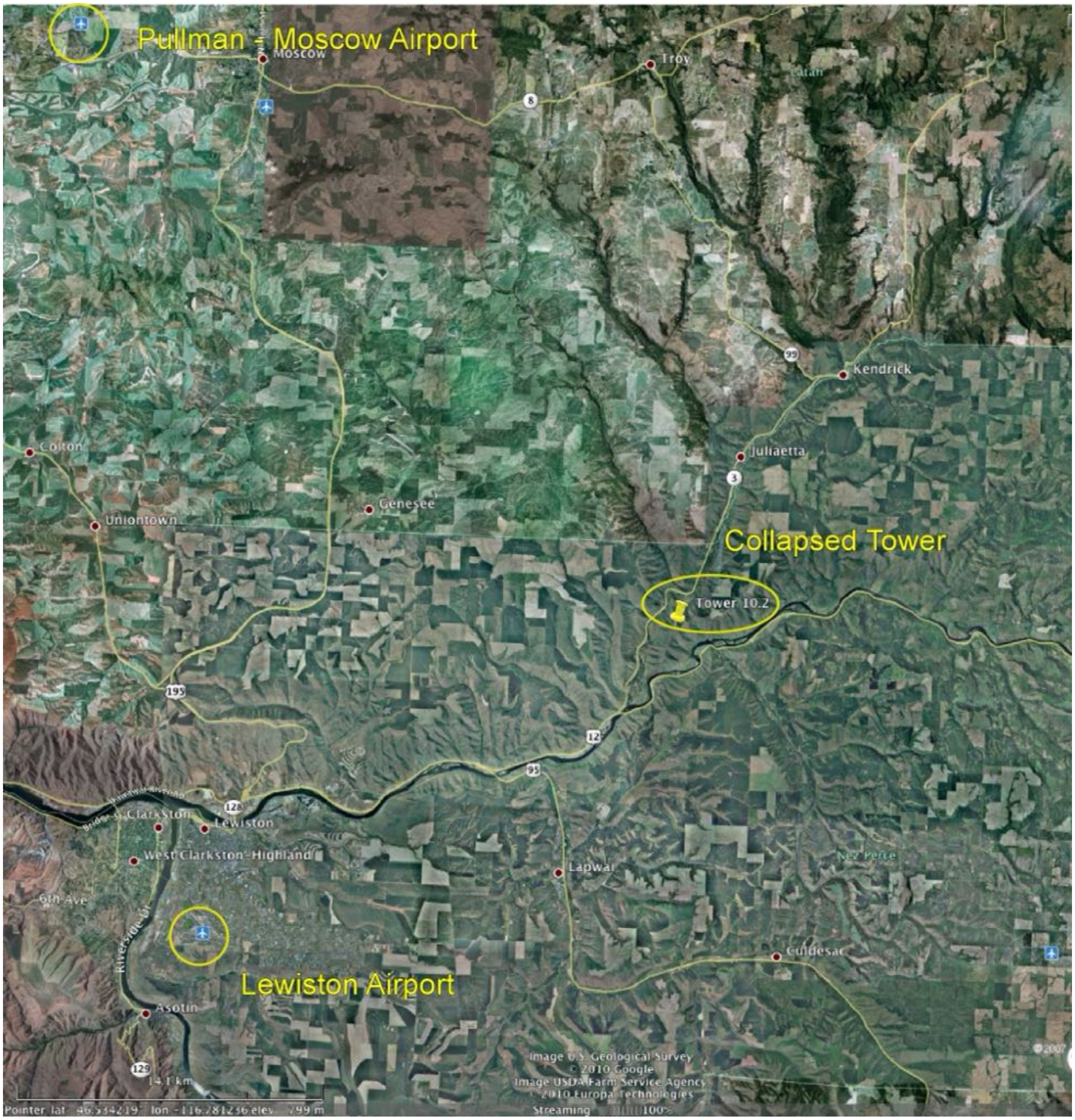

Figure 4-11. Local Map

The recorded weather data from the airports is listed in Table 4-2 and graphed in Figures 4-12 and 4-13.

Airport	Maximum Gust Speed (mph), direction	Time of Maximum Gust	Maximum Sustained Wind Speed (mph)	Time of Maximum Sustained Wind	Precipitation for Nov. 16 (inches)
Lewiston, ID, KLWS 11/16	63, WSW	1:56 am Nov 16	48		0.04
Pullman	85, W	1:24 am Nov 16	47		0.37

Table 4-2. Recorded Weather Data, Nov. 15, 16, 2010

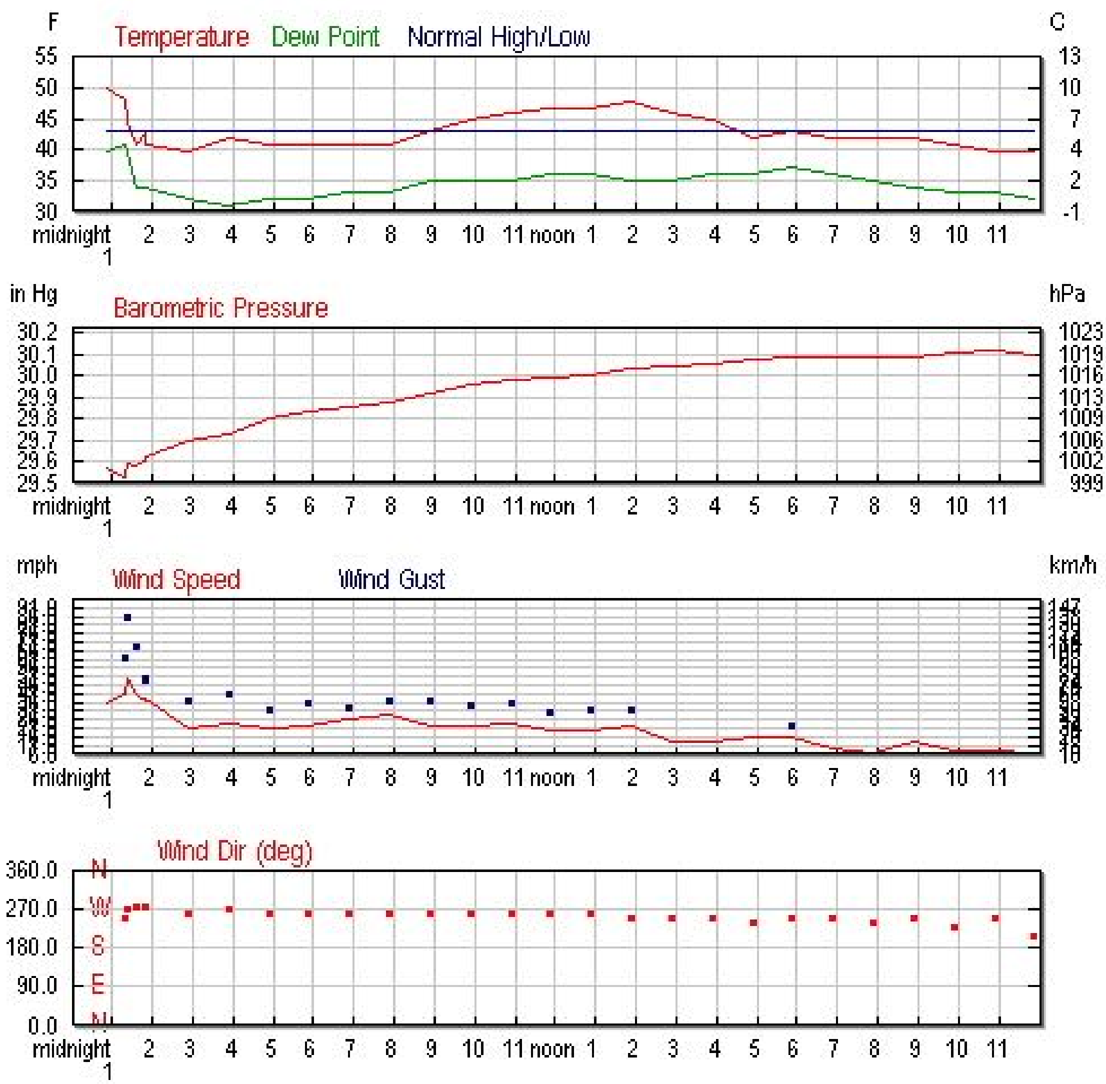

Figure 4-12. Pullman, Weather Data, Nov 16, 2010

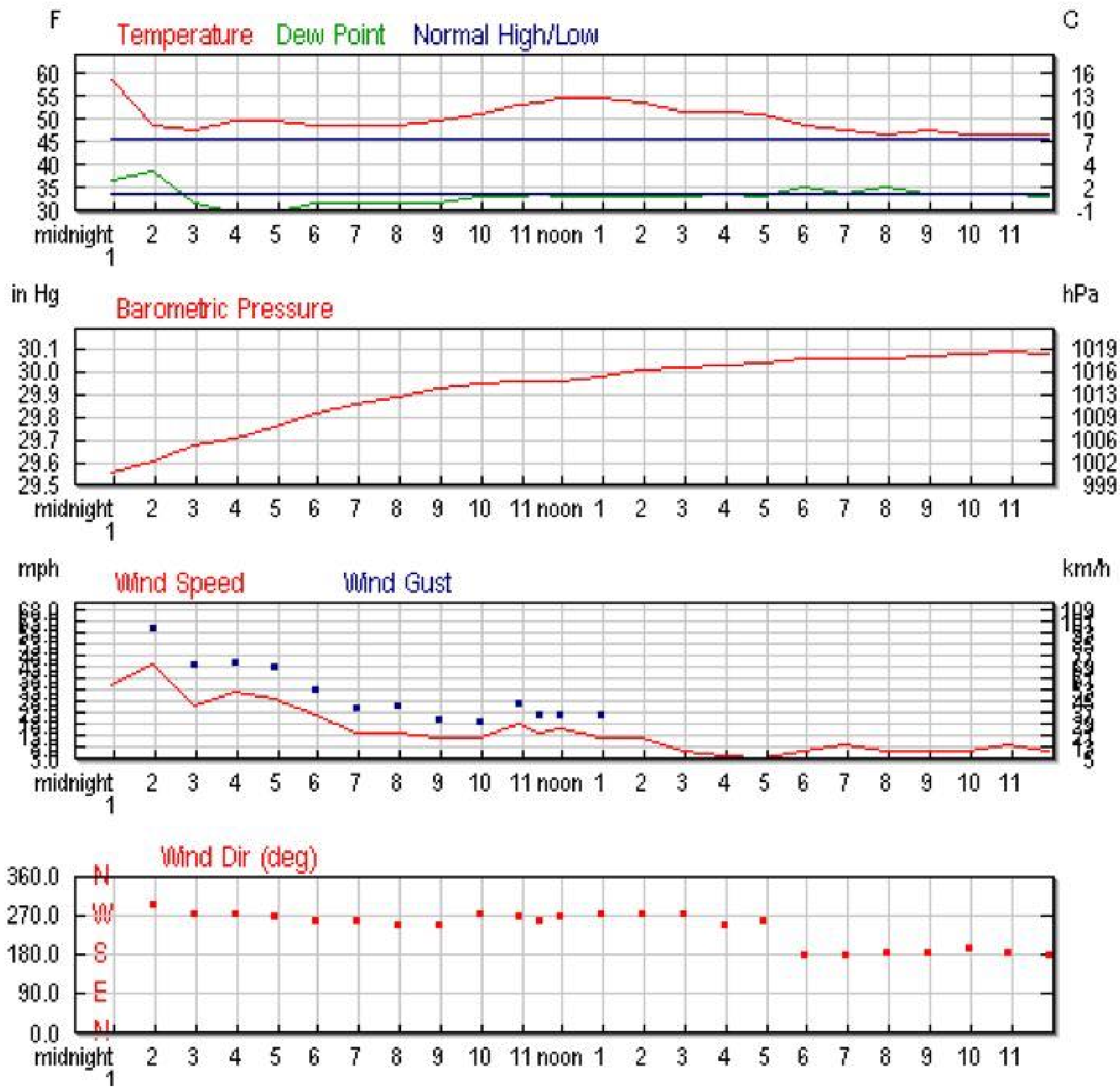

Figure 4-13. Lewiston, KLWS Weather Data, Nov 16, 2010

4.4 Level 1 Analysis

We performed a Level 1 analysis for the Hatwai-Dworshak line. Three "Level 1" analyses were performed (Cases 101, 102, 103), and are included in Excel file Hatwai-Dworshak 0.xls.

Case 101. Case 101 has the following features:

- There are 131 towers in the model, beginning at the Hatwai substation to the west, and continuing to the Dworshak substation to the east.
- We assume a wind speed of 80 mph at Hatwai, reducing to 60 mph at Dworshak. The 80 mph value is based on the two gust readings (63 mph at Lewiston, 85 mph at Pullman, Table 4-2). The decision to reduce the wind speed from west to east is possibly arbitrary, as there are no weather station data available near Dworshak.
- Figure A-6 (BPA wind map, 1980) shows no particular wind gradient in the section of Idaho near where the tower failed, and the entire area is set in Figure A-6 at 60 mph for a 50-year return period wind (Exposure C without topographic factors). There is also no specific wind gradient suggested for a severe Pacific storm over Idaho (Figure A-7).
- Case 101 reflects the same assumptions as Case 8 for the Satsop-Aberdeen No. 2 line. This includes: Beta wind = 0.4, Beta tower = 0.5 (likely too high), D/C cutoff = 0.50 (likely too low), and Kzt = 1.0 (ignores topographic effects).

The key results for Case 101 are in Table 4-3. This analysis suggests 6.44 tower failures, and the chance of failure of structure 10/2 (ObjectID 53462) is 0%. We conclude from this Case 101 analysis that is it not realistic, with the following weaknesses:

- Tower Type 28MV1 is listed as having strength capacity of 127 mph (Column AE in file Hatwai-Dworshak 0.xls, Fragility sheet). This value of 127 mph was based on structural analyses performed in 2008 – 2009. Subsequently, additional information has suggested that type 28MV1 is a particular weak design from the 1960s, and likely has a capacity less than 127 mph.
- Tower 10/2 (ObjectID 53462) is located atop a hill (Figure 4-4), and should have had substantial wind speed up due to topographic factors. Case 101 excludes topographic effects.

Case 102. Case 102 is run, using the same assumptions as Case 11 for the Satsop – Aberdeen No. 2 line. Summary results in Table 4-3 still do not show good correspondence.

In examining the use of slope as a proxy for topographic factors in Case 102, we see that structure 10/2 was assigned Kzt = 1.00, as its local slope (17.28%) is not too steep. However, Tower 10/2 is clearly atop an exposed ridge and is very likely exposed to wind speed up due to topographic effects.

Case 103. A more refined proxy for Kzt is suggested:

- First, for each tower, compare its ground elevation versus those of its westerly neighbor (in the general direction from where the wind is coming from).

- For towers with a positive slope only (i.e., the easterly tower is at higher elevation than the westerly tower), Kzt is set to 2.0 (local slope higher than 0.33) or 1.25 (local slope between 0.20 and 0.33). Kzt is set to 1.00 for all towers with local slope less than 0.20 or negative (i.e., in gentle rises, or in the lee of a ridge top).

- The results are included in Case 103. While the number of tower failures is high (2.23), it is within "credible bounds", and the actual tower that did fail is assigned one of the highest chances of failure (23.2%).

Case	Level	Wind Speed MPH	Beta Wind	Beta Tower	D/C Cutoff	Tower Failures (initiations)	Towers with p(fail) > 10%	Towers with p(fail) > 25%	Tower 53462 p(fail)
101	1	80 – 60	0.40	0.50	0.50	6.44	29	10	0.000
102	2	157 – 60	0.28	0.10	0.75	8.71	21	14	0.000
103	2	148 – 60	0.28	0.10	0.75	2.23	7	3	0.232
104	2	80 – 60	0.28	0.10	0.75	3.11	7	5	0.526
105	2	60 – 60	0.28	0.10	0.75	1.61	4	4	0.268

Table 4-3. Parameter Study, Hatwai – Dworshak

Case 104. Based on discussions with BPA, it is now felt that the original structural calculation of the strength capacity of tower 10/2 (tower type 28MV1) of 127 mph is too high. A more realistic strength capacity for 28MV1 is 100 mph.

In order to preserve each case for this report, Case 104 is performed in a new Excel file, called Hatwai-Dworshak 1.xls. In this file, the strength level for 28MV1 is changed from 127 mph to 100 mph (see column AE in sheet Fragility).

With this adjustment, the model shows an increased chance of tower failure of structure 10/2 to 0.526.

Case 105. For Case 105, we begin with Case 104, but assign a constant 60 mph as the open field exposure C wind speed everywhere along the line. This has the effect of reducing the total tower failure initiations to 1.61 (reasonable) while highlighting the weak tower that did fail (10/2) at 26.8%. Results from Case 105 are shown in Figure 4-16.

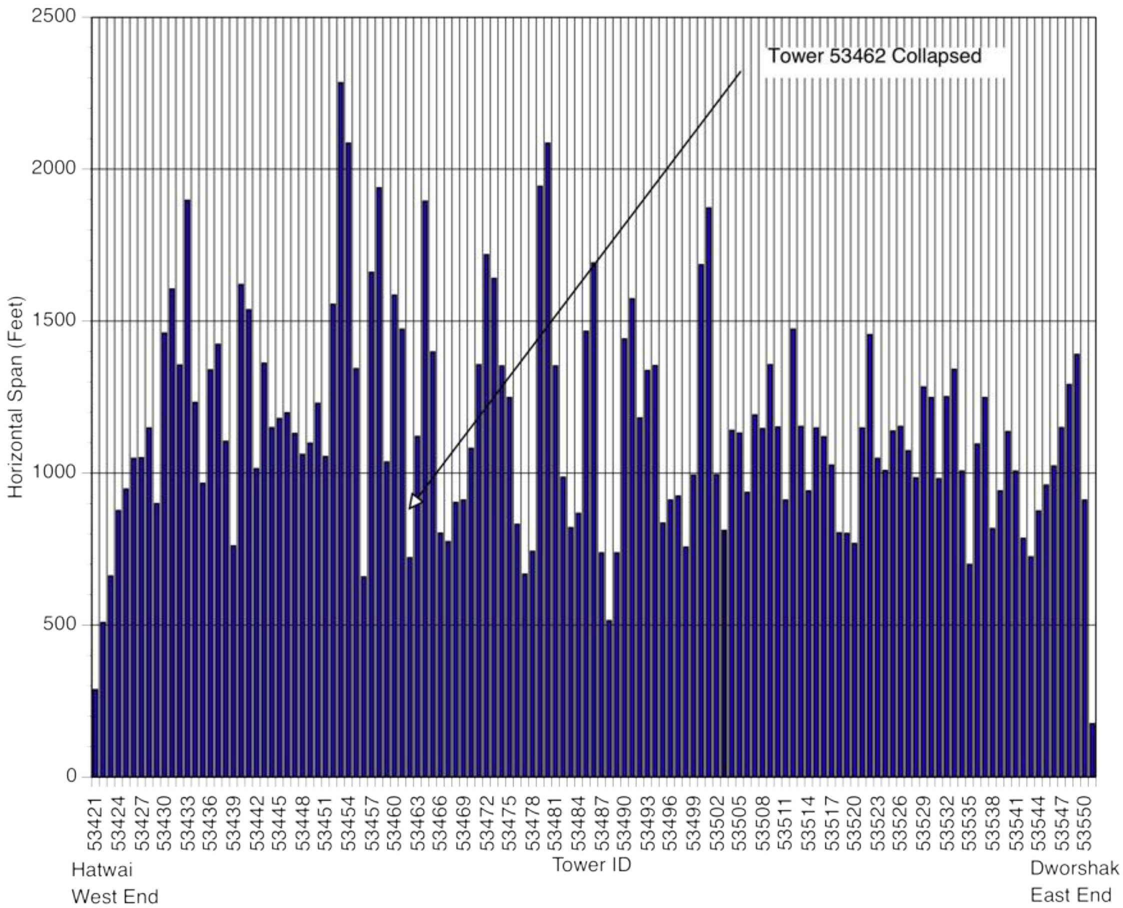

Figure 4-14. Horizontal Spans, Hatwai – Dworshak

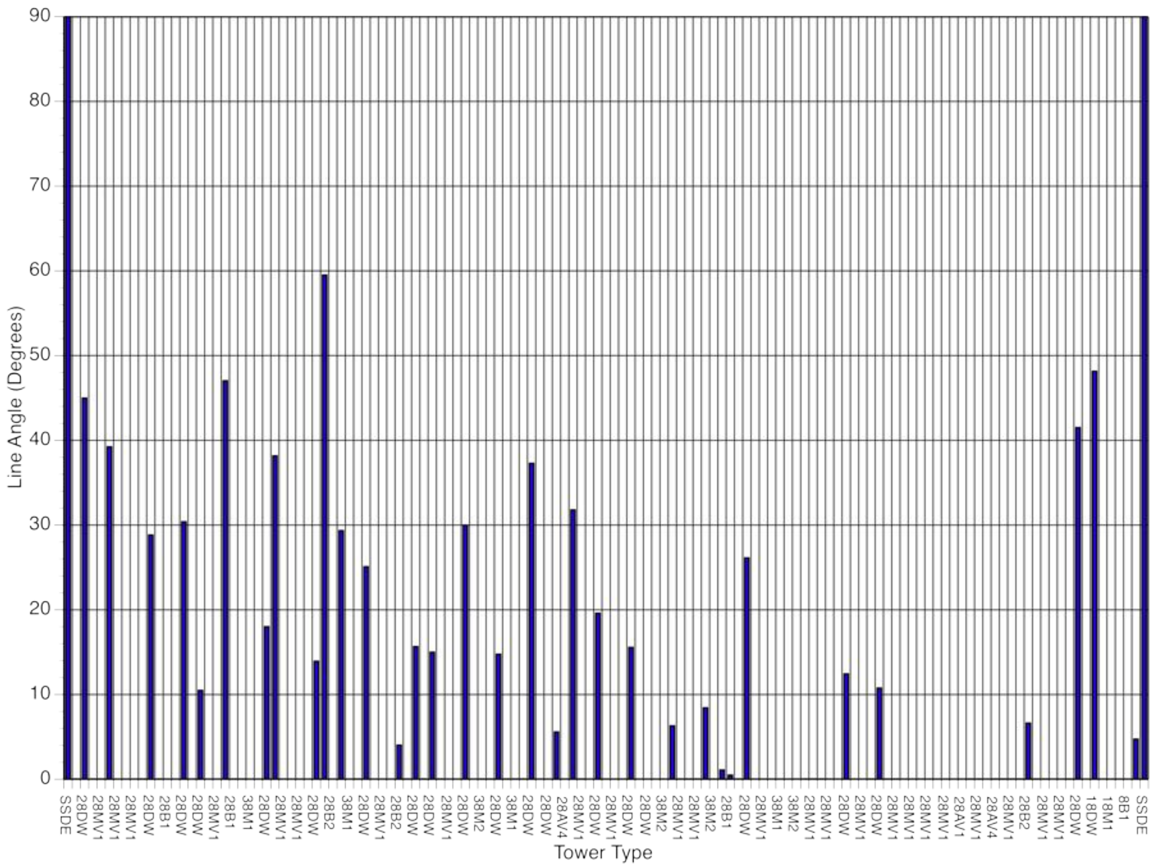

Figure 4-15. Tower Types and Line Angles, Hatwai – Dworshak

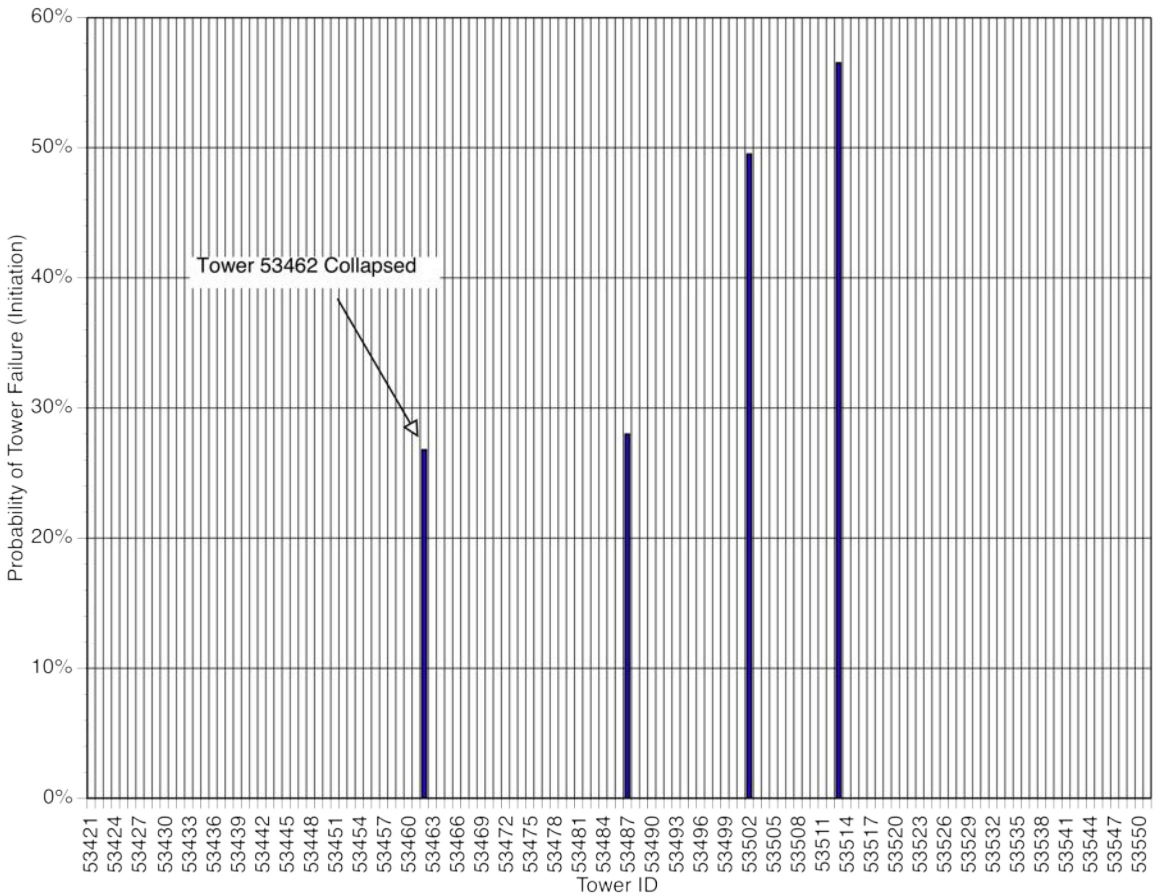

Figure 4-16. Tower Initiation Failure Rates, Hatwai – Dworshak

For Case 105, the four towers most at risk are:

- 53462. Type 28MV1. P(fail) = 0.2678. Kzt = 2.0. Local wind demand = 120 mph. Local capacity = 144.53 mph (due to short span)
- 53487. Type 28MV1. P(fail) = 0.2798. Kzt = 2.0. Local wind demand = 120 mph. Local capacity = 142.96 mph
- 53502. Type 38M1. P(fail) = 0.495. Kzt = 2.0. Local wind demand = 120 mph. Local capacity = 120.45 mph
- 53513. Type 28MV1. P(fail) = 0.5651. Kzt = 2.0. Local wind demand = 120 mph. Local capacity = 114.24 mph

For all these four towers that are predicted to be "weak", the estimated effects due to local topography control the result. While all three other towers are also located along windward facing slopes with high local gradient (see Figures 4-17, 4-18, 4-19), not all are as exposed as 53462 that did collapse, and all three are more parallel to the likely prevailing wind direction, and thus would have seen somewhat lower applied wind forces.

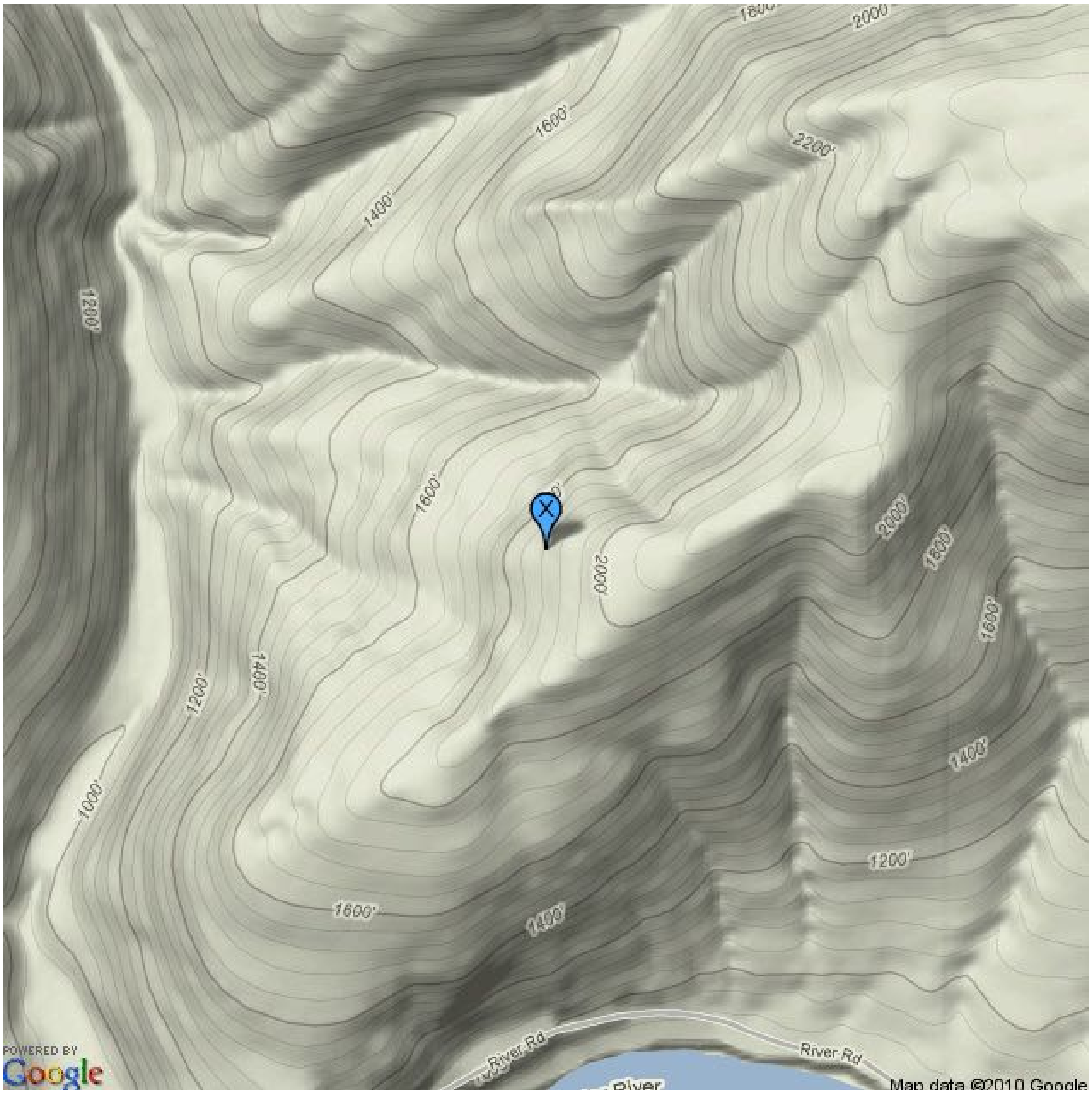

Figure 4-17. Topography, Tower 53487

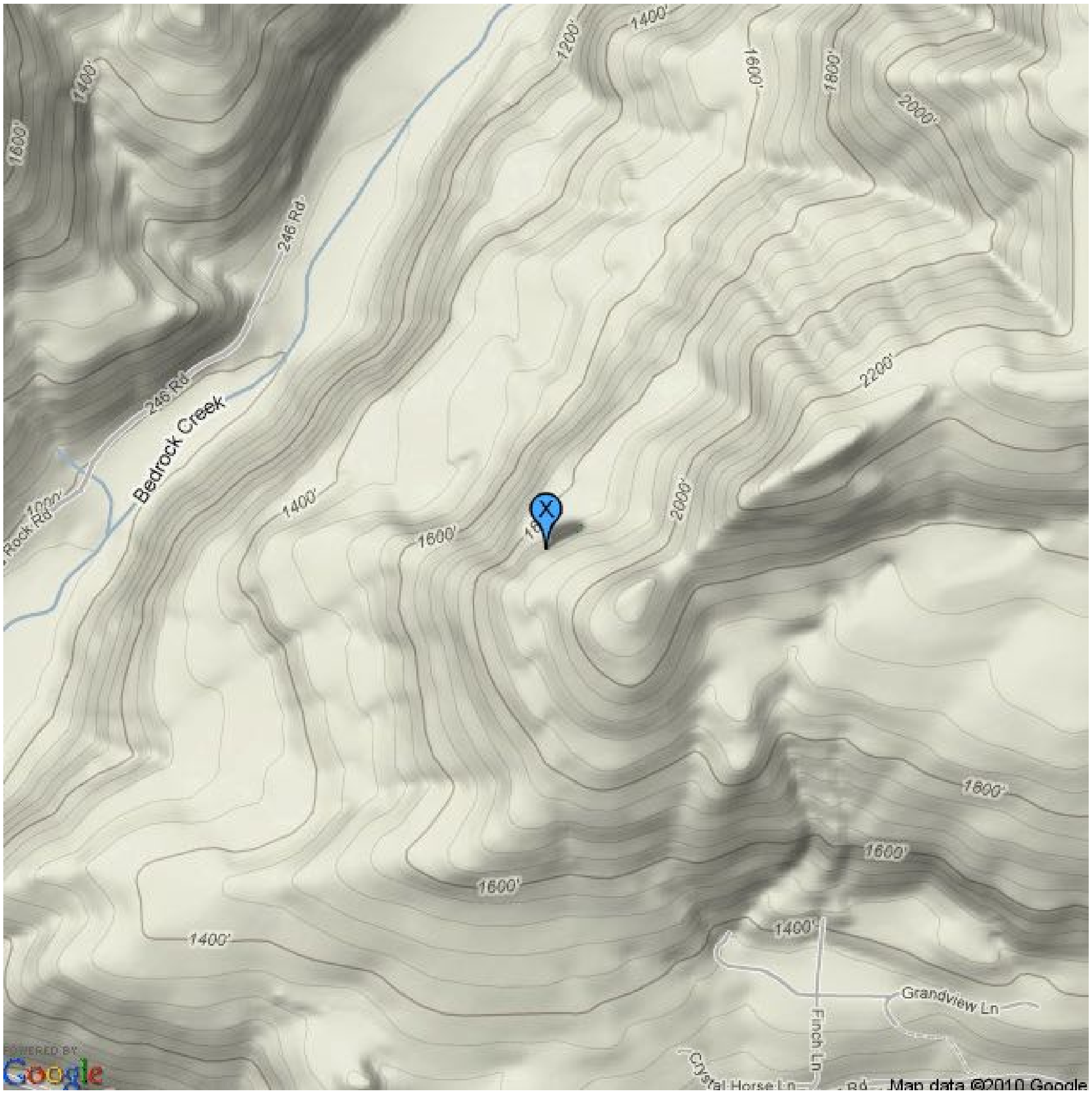

Figure 4-18. Topography, Tower 53502

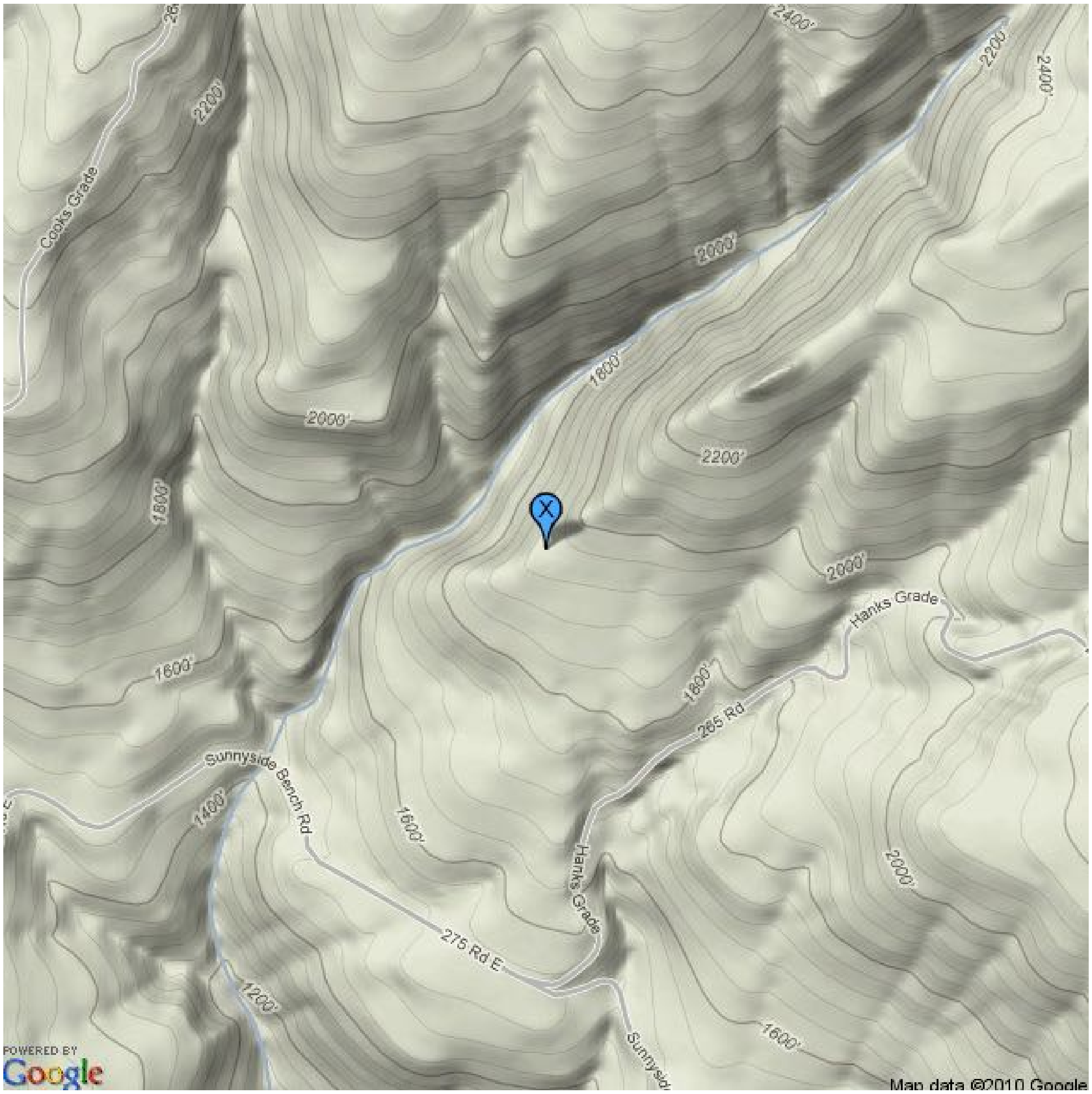

Figure 4-19. Topography, Tower 53515

Conclusions. The primary reasons that tower 10/2 (ID 53462) collapsed are as follows:

- It was located atop a ridge with a topographic wind speed up for winds from the west / northwest.
- Its actual transverse span was about 720 feet. This is substantially less than the design span (1,200) feet for this type of tower. Thus, if one only uses design span/ actual span as a predictive factor for tower collapse, one would never identify tower 10/2 as being problematic.
- The storm likely produced wind gusts on the order of 60 mph in open terrain, and on the order of 120 mph± (likely range 90 to 140 mph) at the ridge top with tower 10/2. While Case 105 shows a wind speed at tower 10/2 of 120 mph, due to the lack of nearby

anemometer data, there is no way to know with any certainty as to what the true wind speed was.

- The tower design for 28MV1 is perhaps able to resist about 100 to 120 mph as a transverse wind, for a 1200 foot span.

The other three towers shown to be highly loaded (Case 105) did not actually collapse, in part because the prevailing wind direction was more-or-less parallel to the lines. Had the wind been from the north (not the likely prevailing wind direction), these towers might have collapsed.

5.0 SAMPLE ANALYSIS 3: ICE EVENT

In Section 5, we examine the failure of a 345 kV circuit in New Brunswick, due to ice loads.

The description of this transmission line is based on data provided to us by the New Brunswick Power Transmission Corporation (NBPTC). NBPTC is a subsidiary of NB Power. NBPTC's transmission grid includes 6,801 km of 138 kV, 230 kV and 345 kV transmission lines.

The available NBPTC data is not in the same format or at the same level of detail as the data used for the two examples in Sections 3 and 4 for the BPA system. The reliability analysis is adjusted to accommodate the differences.

5.1 Location of Circuit and Collapsed Towers

A storm with sleet, freezing rain and light winds occurred over a 48 hour period from April 2 to April 3 1993 in Southern New Brunswick. This resulted in heavy ice loading on transmission and distribution circuits in the area. At 10:04 am on April 3 1993, NBPTC's 345 kV Line 3002 failed. The failure included 15 transmission towers, of which one was a heavy angle (dead-end) tower, and 14 were suspension tangent-type towers. Of these 15 collapsed towers, arguably 13 (maybe 14) of the towers were due to cascading effects (zippering), and at most 1 (maybe 2) were due to overloads.

Campbell (1995) describes the transmission line as follows (see Figure 5-1).

A guy wire pulled apart on structure 294 (tangent suspension tower). This allowed the tower to rotate. This rotation, coupled with ice loading, caused adjacent dead-end structure 295 to collapse. The failure of dead-end structure 295 plus 294 resulted in cascading failures of 3 additional suspension towers towards the north (293, 292, 291) and 10 additional suspension towers towards the south (296, 297, 298, 299, 300, 301, 302, 303, 304, 305). Figures 5-1 and 5-2 show the plan and profile configuration of Line 3002 in the vicinity of the failures.

Figure 5-3 shows the typical arrangement for the failed dead end structure (295). Figure 5-4 shows the typical arrangement for the failed guy-wire tangent suspension towers (291, 292, 293, 294, 296, 297, 298, 299, 300, 301, 302, 303, 304, 305). The adjacent towers that did not fail (290, 306) are the same as Figure 5-4. Using NBPTC nomenclature, the dead-end tower (295) is type "D", the initiation tower (294) is type "B", and the remaining towers are type "A".

Both these types of towers are in widespread use for the 345 kV transmission network in New Brunswick.

The conductors on the line were 795 kcmil Drake ACSR, bundled using two conductors per phase. Drake conductors are specified as having an outside diameter of 1.108", unit weight of 1.094 pounds per foot, and with rated strength of 31,500 pounds. Phase spacing is 9 m (Figure 5-3) or 10 m (Figure 5-4).

The ground wires on the line were 7/16" diameter. It is assumed that the ground wires are steel, have an outside diameter of 0.435 inches, unit weight of 0.398 pounds per foot, and a rated strength of 14,500 pounds.

Standard 146 mm (5.75") diameter porcelain insulators with a mechanical strength of 111.2 kN (25,000 pounds) were used in the suspension insulation strings of these towers (Figure 5-4) and doubled insulator strings for the dead end tower (Figure 5-3).

Original design information for the towers were not provided to the developers of this report. To perform the reliability analysis (Section 5.4), certain assumptions are made about the original design loadings for the glaze ice condition, as well as mechanical characteristics of the towers and their guy wires.

Campbell reports that the failure initiation was due to tension separation failure of a preformed guy splice on structure 294.

All the structures south of 295 fell towards the south; and all the structures north of 295 fell towards the north.

Ice build up on the conductors varied in thickness and configuration, depending on location along the line. The equivalent of 25 mm (1") of radial ice was observed on some of the conductors, and nearly 50 mm (2") on one face of the conductor was reported in one location. Post-mortem calculations for structure 295 suggest that at 1" of ice and no wind, the structure was at 75% of its strength capacity.

Adjacent 345 kV Lines 3003 and 3004 (Figures 5-1, 5-7) experienced nearly the same ice and wind loading, but had no failures.

From field observations, of the six bundles of conductors (12 conductors total, six on north side and six on south side) and four ground wires (2 on north side and 2 on south side) that were attached to the collapsed 295, the following occurred after the bridge of 295 hit the ground:

- The turnbuckles that attached the three 2-bundle conductors on the south side of 295 all disengaged. This would release the full tension force in the conductors onto adjacent tower 296, leading to 10 towers that cascaded to the south.

- The turnbuckles that attached the three 2-bundle conductors on the north side all held. This may be a reason to explain the relatively fewer towers (4) that cascaded to the north.

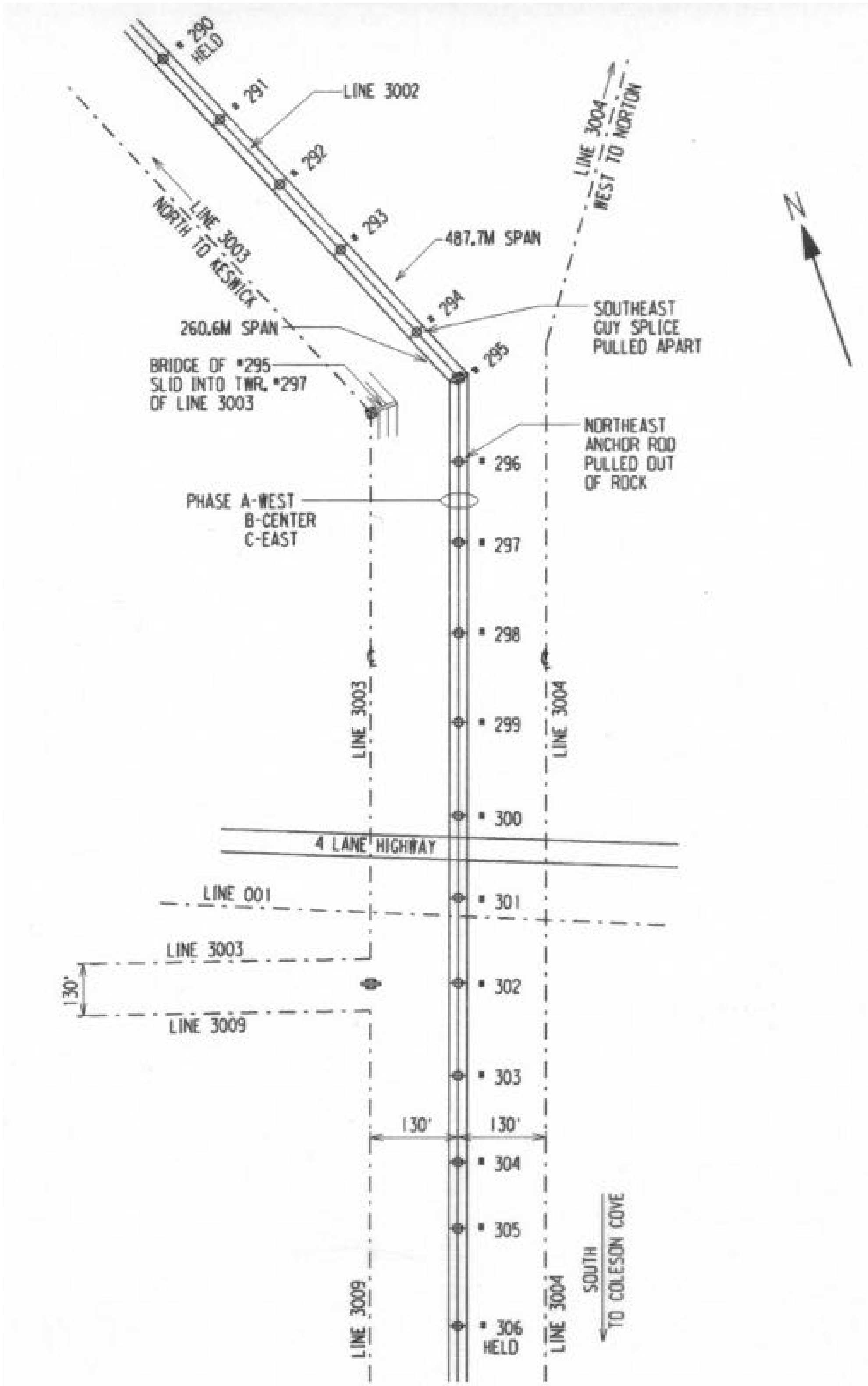

Figure 5-1. Line 3002 – Plan (Not to scale)

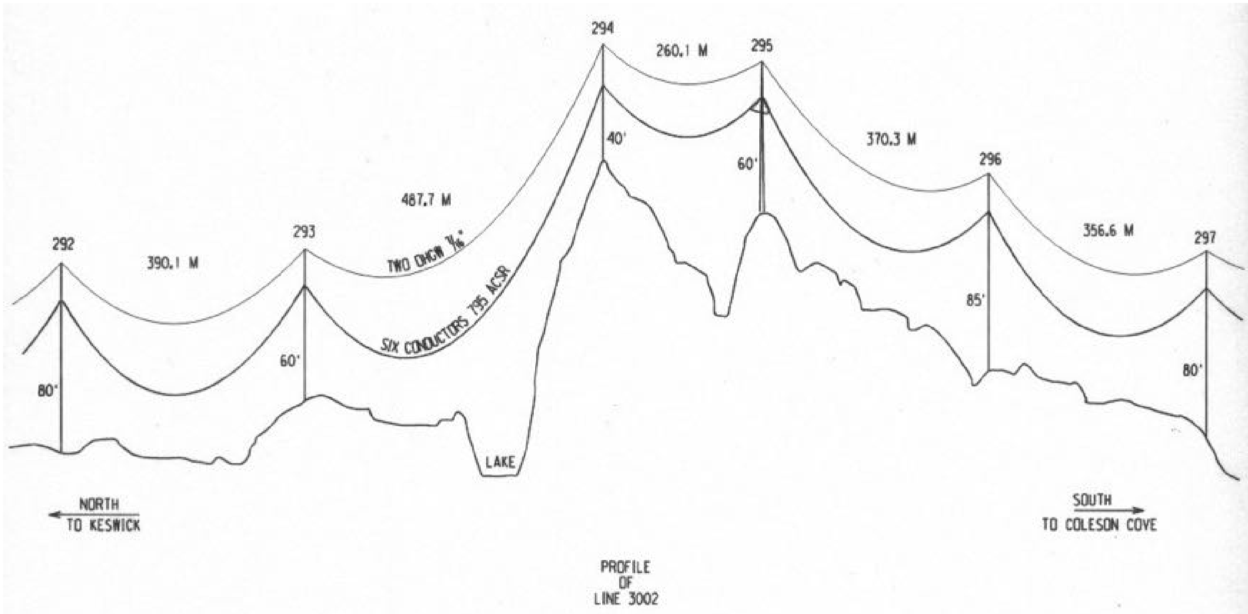

Figure 5-2. Line 3002 – Profile

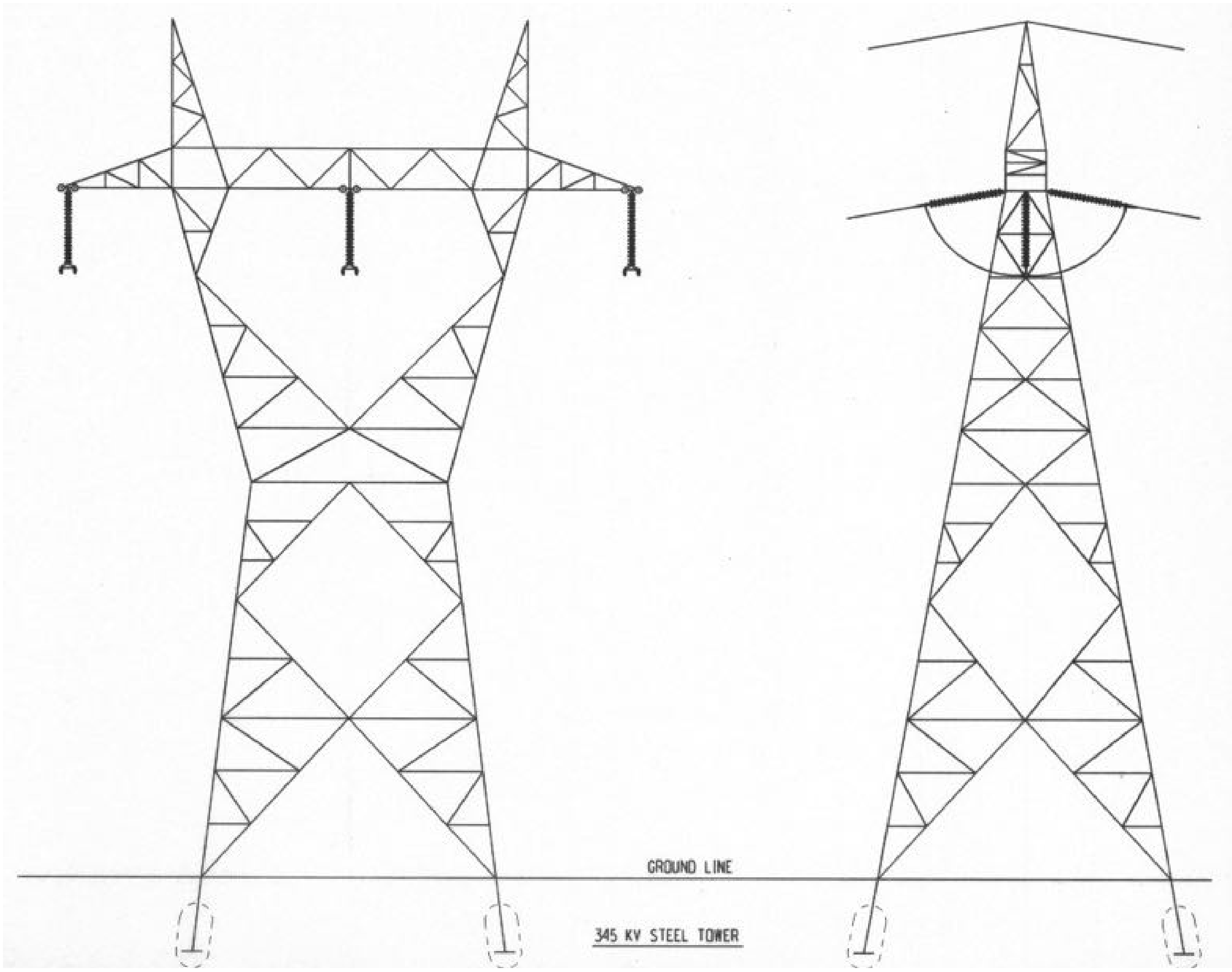

Figure 5-3. Structure 295 (Dead End Tower)

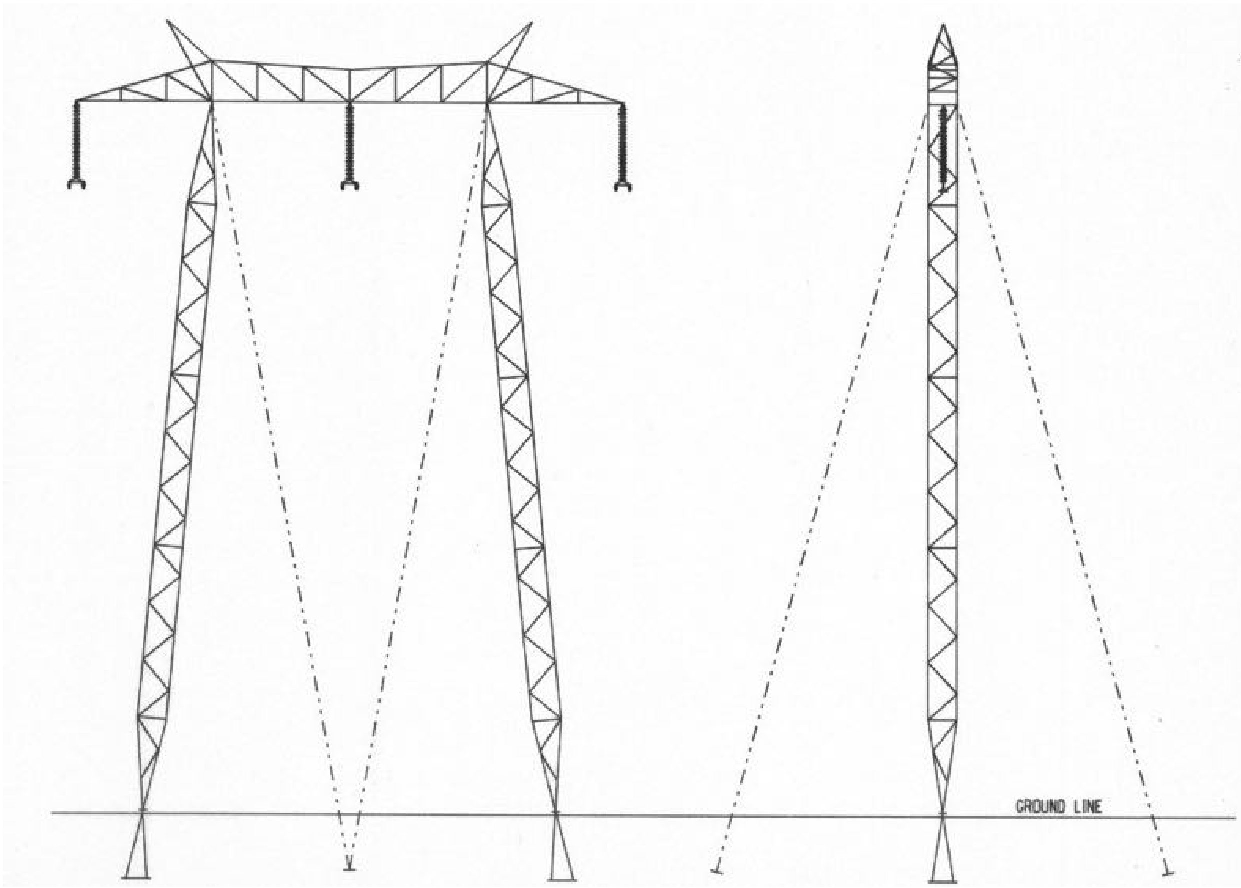

Figure 5-4. Structure 291-294, 296-305 (Guyed Tangent Towers)

Figure 5-5 shows an aerial view of a portion of Line 3002. Line 3002 is highlighted; the shaded zone delineates the collapsed portion of the line; the black circle shows tower 295; the black lines delineate the collapsed suspension towers (not to scale).

Figure 5-5. Location of Failure (Shaded zone). Towers shown at approximate locations

NBPTC provided a file with latitude and longitudes for each tower along Line 3002. This data was converted from latitude – longitude into projected units (meters) assuming the earth is a perfect globe of radius 6,371 km. While this projection is not sufficiently accurate for modern land survey purposes, it is accurate enough to obtain distances between towers for purposes of the reliability analysis. This conversion from lat-long to eastings-northings is provided in file L3002 LatLong_km.xls.

Analysis of the projection of the tower locations shows the following:

- L3002 has a length between substations of 116.694 km (72.497 miles). This matches NBPTC's reported length of the circuit of 116.7 km.
- The average horizontal distance between towers is 366.96 meters (1,204 feet)
- The maximum horizontal distance between towers is 562.16 meters (1,843 feet)
- The average horizontal span per tower is 366.89 meters (1,203 feet)
- The maximum horizontal distance between towers is 470.31 meters (1,543 feet)
- Dead end towers have a range of angles up to 48 degrees

Table 5-1 provides the spatial statistics for the towers near the collapse. Based on the latitudes and longitudes of the towers, the spans between towers (Figure 5-2, based on 1974-vintage design plans and profiles) differ somewhat from the current (2011-GIS-coded) locations. The reason for this discrepancy is unknown; it is possible that some rebuilt towers (as indicated in the current location data) were placed at somewhat different locations than the collapsed towers.

Tower	Type	Status	Horizontal Span Ahead (2011) (m)	Angle Change (2011) (°)	Reported Span Ahead (1974) (m)	Reported Angle (1974) (°)	Horizontal Span at Tower (2011) (m)
290	A	OK	333.52	1.26			323.61
291	A	Zipper	399.57	0.41			366.54
292	A	Zipper	389.91	-1.63	390		394.74
293	A	Zipper	484.76	0.97	488		437.33
294	B	Initiation	254.57	-0.22	260		369.66
295	D	Zipper	402.39	-43.55	370	43	328.48
296	A	Zipper	319.99	2.90	357		361.19
297	A	Zipper	402.90	-1.22			361.44
298	A	Zipper	388.56	-1.02			395.73
299	A	Zipper	415.28	0.68			401.92
300	A	Zipper	353.71	0.37			384.49
301	A	Zipper	373.12	-0.50			363.41
302	A	Zipper	399.22	0.10			386.17
303	A	Zipper	380.49	-0.10			389.85
304	A	Zipper	295.82	0.01			338.15
305	A	Zipper	419.97	0.05			357.90
306	A	OK	310.55	-0.05			365.26

Table 5-1. Tower Statistics Near the Collapse

Campbell concluded the following:

- The separation of a guy wire splice on tower 294 caused the line to fail. Post-mortem structural analyses suggested that the guy wire should have been at about 75% of its rated strength (neglecting the splice), if there had been 1" of radial ice (with no wind) on the line.

- Adjacent Lines 3003, 3004 and 3009 were loaded with similar amounts of snow, ice and wind. These lines did not fail.

Figure 5-6 shows the terrain and locations of towers 295, 294 and 293.

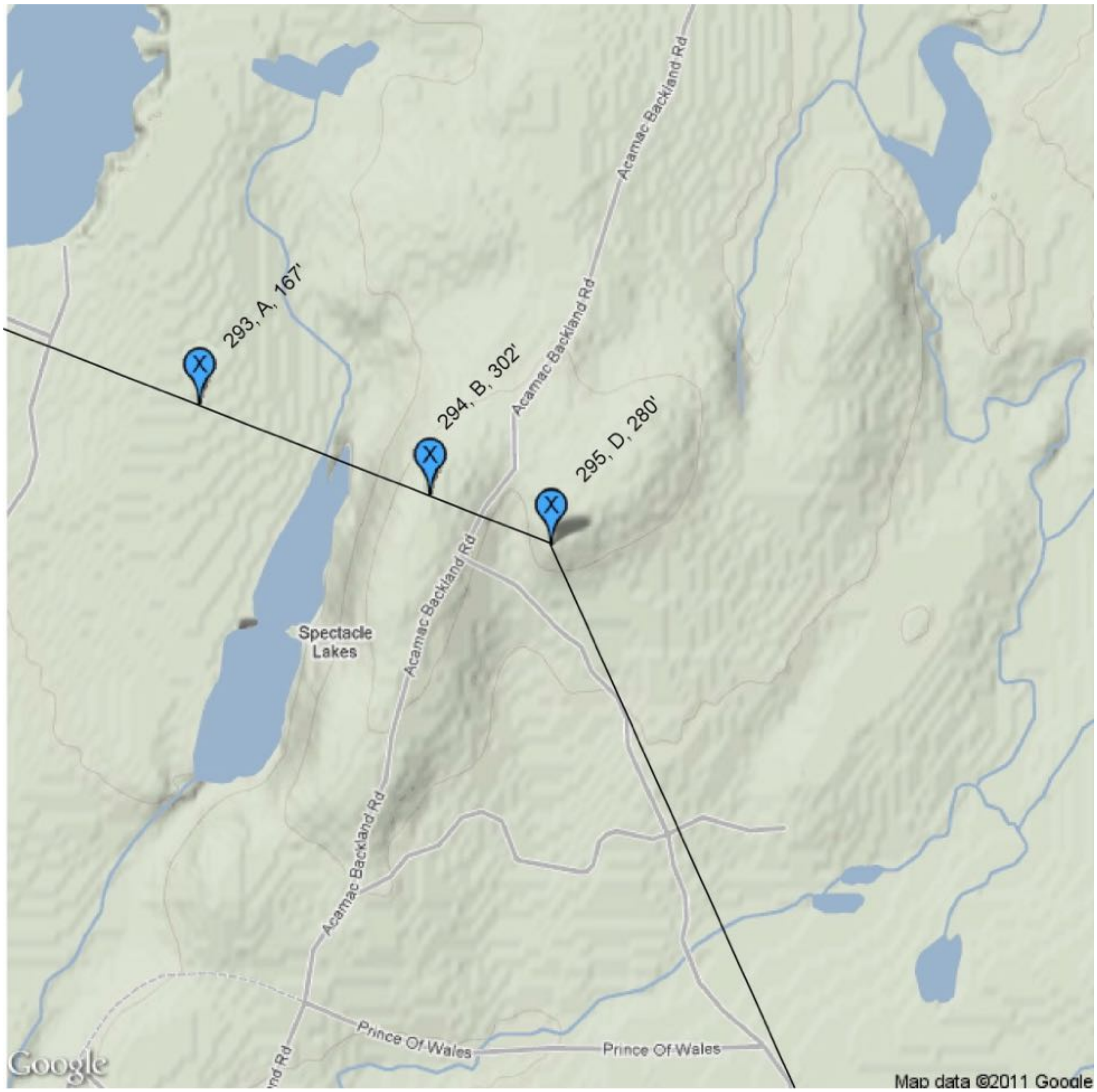

Figure 5-6. Location Towers 293, 294 and 295

Figure 5-7 shows a photo taken in 1993 of one of the failed towers (301). The undamaged tower to the left side of the photo is on the adjacent non-collapsed line.

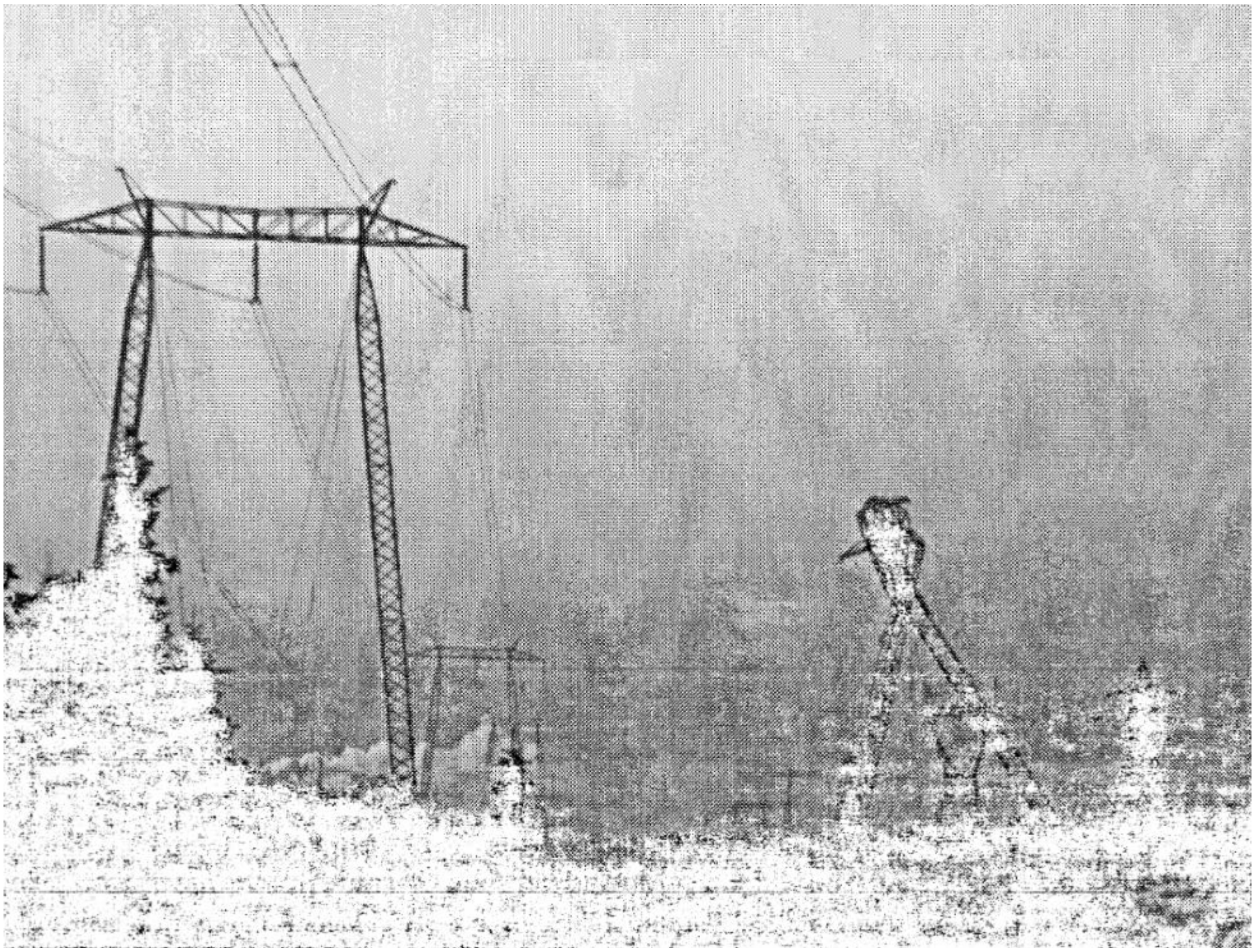

Figure 5-7. Damaged Tower Type A 301 (right side). Undamaged Tower Type A (left side)

Figure 5-8 shows a schematic of the transmission system near the failure of line 3002. While the ice and wind may have initiated failure for structures 294/295 on L3002, the similar ice and wind loads did not damage many nearby similar-style 345 kV structures in Lines 3003 (345 kV) and L3009 (345 kV) and L3004 (345 kV).

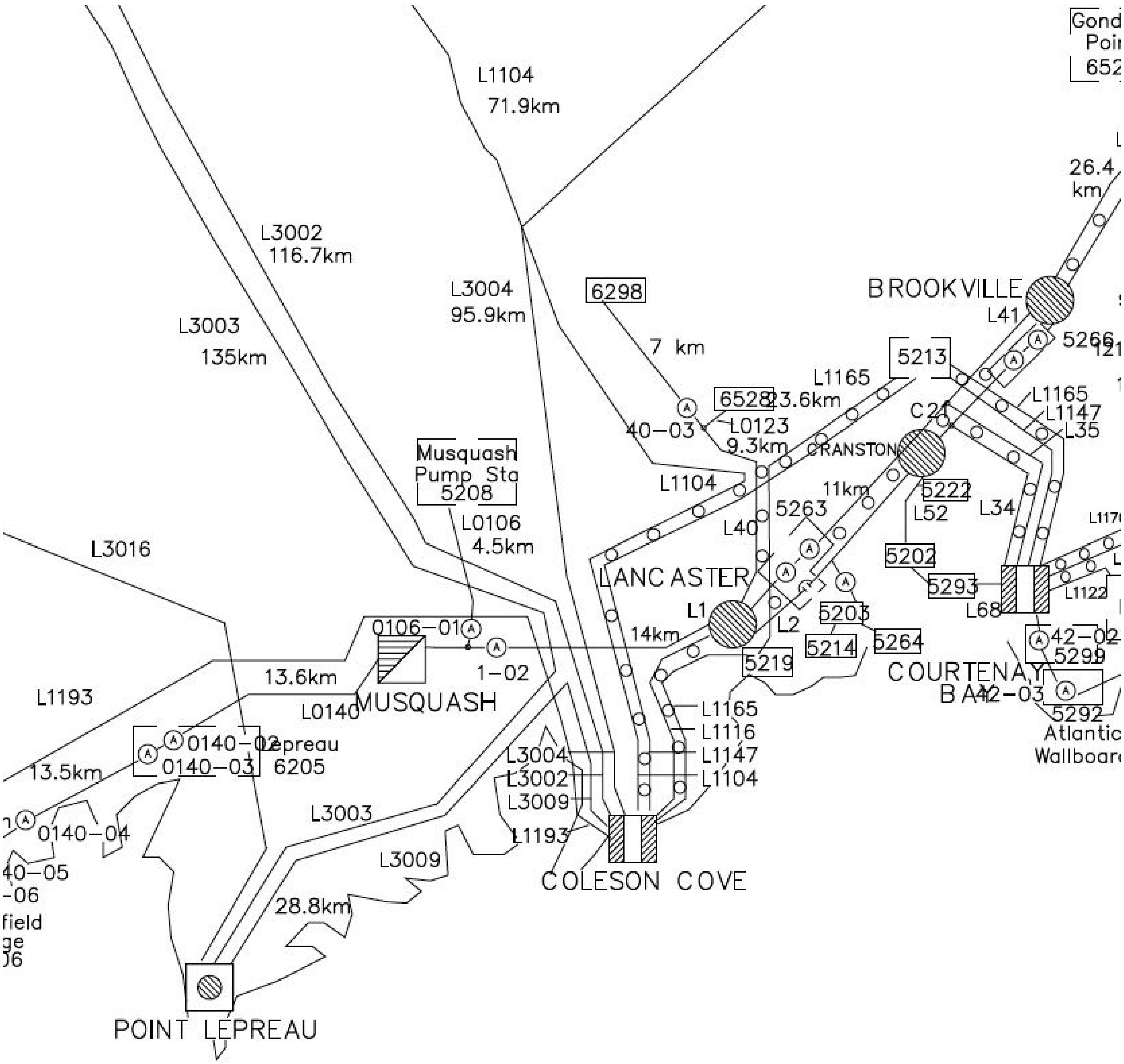

Figure 5-8. Transmission Network Near Line 3002

5.2 Characteristics of the Storm

The nearest weather station is at the airport at Saint John, New Brunswick, station CYSJ Elevation 358 feet. This weather station is located about 30.7 km NE of structure 295 on Line 3002.

For April 2, 1993, the weather report (Figure 5-9) was: mean 28°F, max 30°F, min 26°F; humidity 86 / 89 / 100% (min / average / max). Precipitation 0.22 inches. Wind speeds 12 mph (typical), 18 mph (max sustained), 26 mph (max gust). Events were described as fog, rain, snow, thunderstorms. Prevailing wind was generally from the NNE to NE. Winds from NNE would be roughly perpendicular to the conductors spanning between structures 294 and 295.

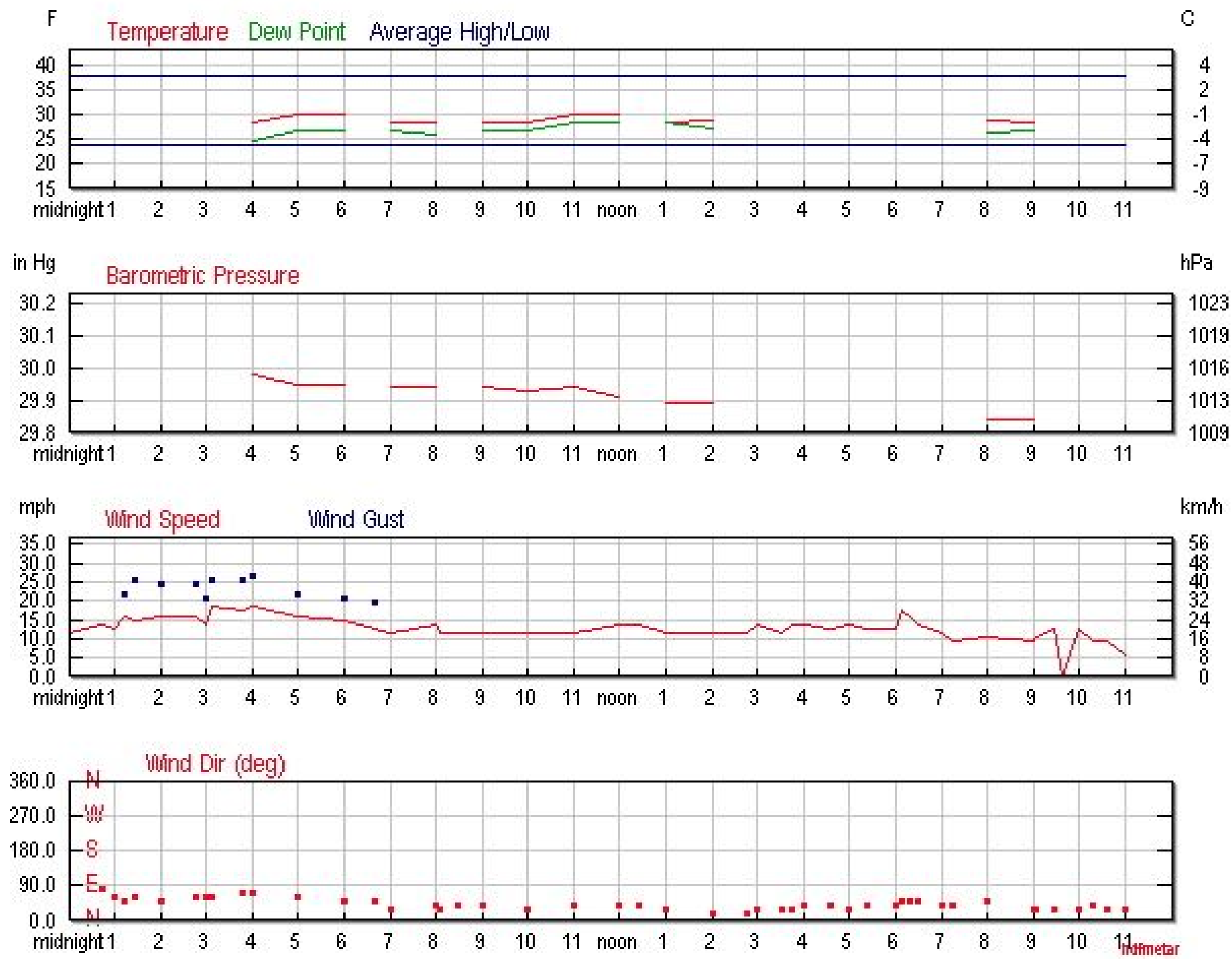

Figure 5-9. Weather, April 2, 1993 (Station CYSJ)

For April 3, 1993, the weather report (Figure 5-10) was: mean 28°F, max 30°F, min 26°F; humidity 86 / 93 / 100 (min / average / max). Precipitation 0.19 inches. Wind speeds 9 mph (typical), 17 mph (max sustained), 22 mph (max gust). Events were described as fog, rain, snow. Prevailing wind was generally from the NNE near the time that L3002 failed.

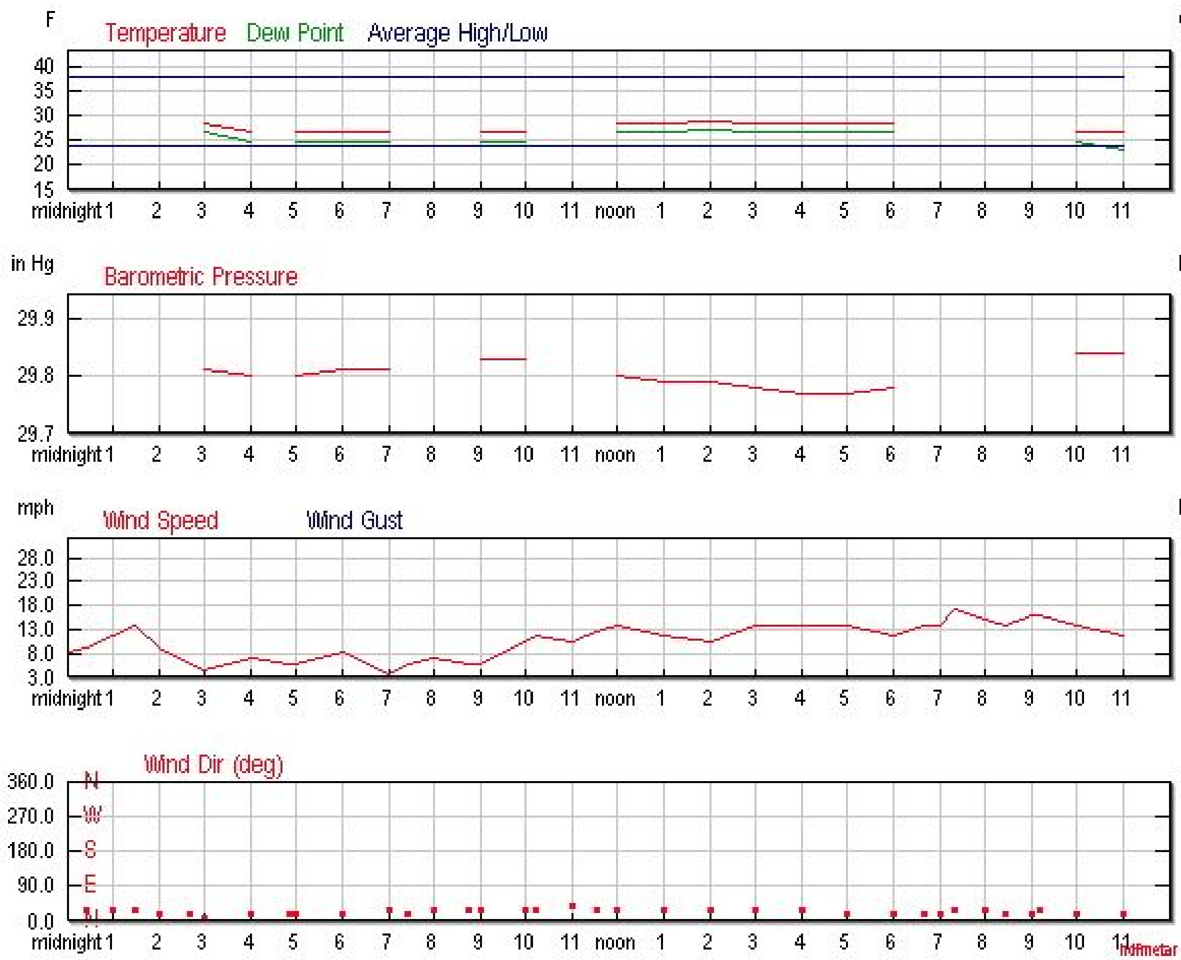

Figure 5-10. Weather, April 3, 1993 (Station CYSJ)

Line 3002 failed at 10:04 am April 3, 1993. At this time, station CYSJ reported winds at 10.4 mph, winds from the NNE. Light freezing rain and light freezing drizzle was reported continuously from 12:24 am April 3 to 11:31 am April 3. Winds were generally 4 to 6 mph during the accumulation of freezing drizzle from 2:40 am through 9:00 am on April 3, but then picked up to 10 to 11 mph between 10:00 am and 10:12 am.

The other nearby weather station is at Point Lepreau, CWPE, located near sea level (20 feet elevation), located about 30 km southwest of the structure 295. There is limited data for this station: April 2, 1993: calm winds from 8 pm to midnight. April 3, 1993: Calm winds all morning long.

5.3 Ice Storms in New Brunswick

New Brunswick has had a history of ice storms. Current wind- and ice-loading maps for the area of New Brunswick with the 1993-failed Line 3002 show the following:

- 50-year winds: 62.5 mph (100 kph) (10-minute) (Figures A-1, A-2)
- Hurricane winds: 71 mph (50 years) to 128 mph (2,000 years), if we conservatively extrapolate for coastal Maine, Figure A-9. If we assume that the site is more like 200 km inland (probably a better assumption), then hurricane winds for L3002 would be about 66 mph (50 years) or 117 mph (2,000 years).
- Glaze Ice. The 50-year return period glaze ice map for Canada (Figure B-1, 2009) suggests that line 3002 is exposed to 25 mm (1") of glaze ice. The extrapolation to New Brunswick from the USA (Figure B-3, 2002) is 1" of glaze ice with 40 mph concurrent wind.
- The NESC Loading Districts for the USA (2002) places Maine in a "heavy" load district; meaning a minimum design of 0.5" radial ice with concurrent 4 psf wind load.

These current wind and ice maps are dated 2002 to 2009, and were prepared well after Line 3002 was placed in service in December 1974. The question arises: what were the wind and ice loading requirements for the actual towers that were constructed circa 1974? Also, as many of the Line 3002 towers may have used "standard" designs, the design basis for these "standard" designs might be from an even earlier time frame. To help sort through these questions, a quick overview of some historic storms in New Brunswick and nearby provinces follows.

Ice storm of 1956. On January 2, 1956, a severe ice storm struck the eastern side of New Brunswick (Figure 5-11). After 10 days of continuous freezing rain and sleet, 423 miles of distribution line and 10 miles of 69 kV transmission line were knocked down. At the height of the storm, 23,000 customers were without electricity.

In nearby western PEI the ice storm damaged 3,500 telephone poles and 2,000 power poles. Based on Maritime Electric records, damaging ice storms have affected PEI in 1954, 1956, 1962, 1976, 1983, 1984, 1988 and 1995.

Figure 5-11. Ice Storm Damage, January 2, 1956

Wind storm of 1976. On February 2, 1976, a gale racked Saint John with one of the fiercest storms ever to hit the Bay of Fundy. At the height of the cyclone, winds were clocked at 188 kph (118 mph).

Ice storm of 1984. Between April 11 and April 14 1984, an ice storm accumulated ice up to 6-inches thick on power lines near Saint John's Newfoundland. There were power outages affecting 200,000 residents of Avalon Peninsula.

Ice storm of 1998. The ice storm of January 1998 resulted in heavy ice loading in Ontario, Quebec and New Brunswick (as well as some parts of the USA). Figures 5-12 and 5-13 show the ice accumulations. In Maine and southern parts of New Brunswick, maximum ice accumulations were reported to be about 40 mm (ice accumulations reported may not correspond to the actual radial ice thickness on transmission lines). The very southernmost section of Line 3002 may have seen as much as 40 mm of ice, with the section near structure 295 seeing about 15-20 mm of radial ice. NB Power (the power distribution company for New Brunswick) reported a cost of $2,000,000 to make repairs. Roughly, 700,000 of 1.2 million people of Maine were without electricity.

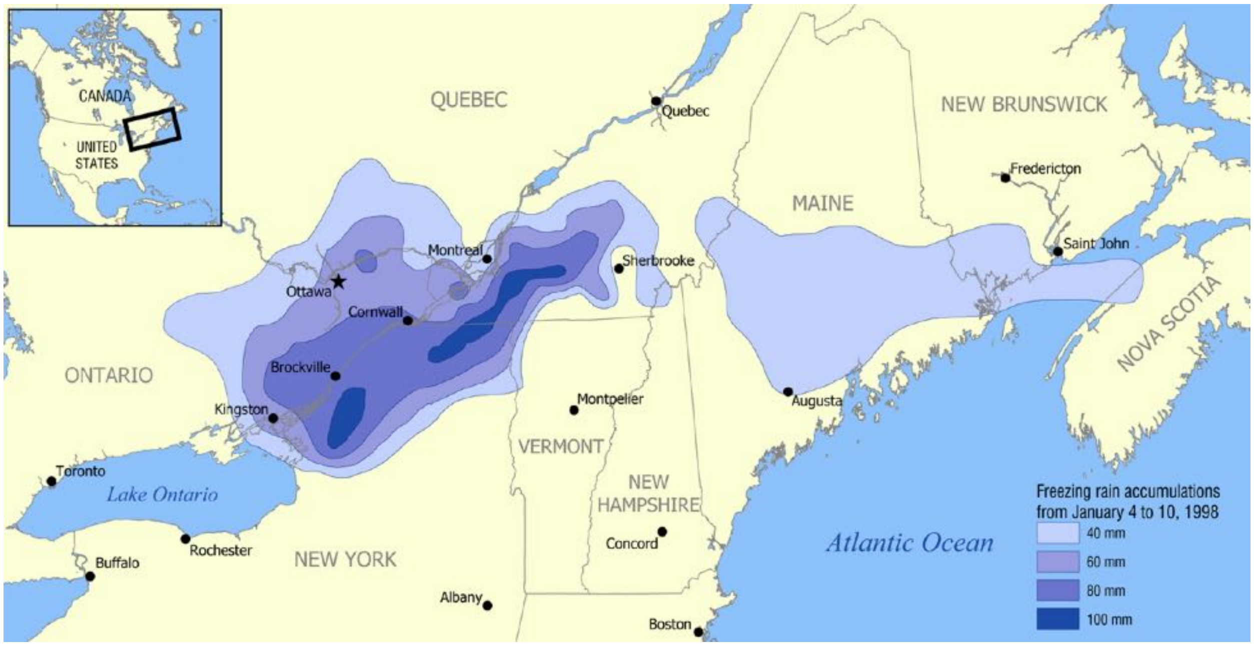

Figure 5-12. Regional Ice Storm Map, January 4-10, 1998 (ref. Environment Canada)

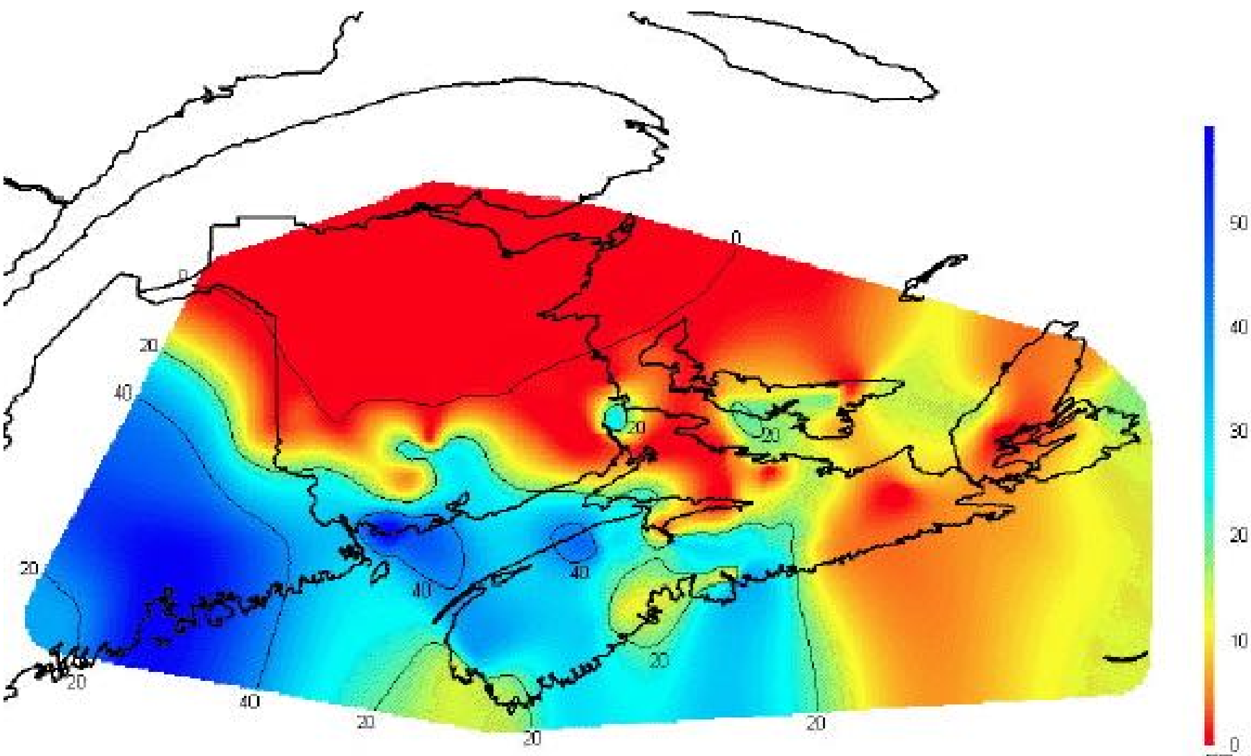

Figure 5-13. New Brunswick Ice Storm Map, January 4-10, 1998 (ref. Environment Canada)

Ice storm of 1999. On February 23, 2991, an ice storm near Cornerbrook (western Newfoundland) resulted in ice covering wires and branches up to 2.75 inches thick. There were power outages affecting 1,600 customers.

Ice storm of 2003. On February 2, 2003, an ice storm coated the eastern part of New Brunswick with up to 58.3 mm of frozen rain (Table 5-2, Figure 5-14). At Wolfe Lake, 58.3 mm of freezing rain fell (~100 km NE of Line 3002). Point Lepreau and Saint John weather stations reported about 10 mm of freezing rain (these two weather stations are about 30 km either side of Line 3002). Damage was worst in Moncton (~140 km NE of Line 3002). Following the storm, the weather turned windy (wind speeds of 75 kph, 47 mph) and very cold (wind chills to -27 C°). Distribution poles snapped with ice reported up to 33 mm coupled with high winds. New Brunswick Power reported it was the worst ever, far exceeding in repair cost and magnitude the 1998 ice storm; cost to repair cables, pole and transformers was put at $3 to $6 million; more than 63,000 customers lost power, with thousand going almost a week without power.

Station	Longitude	Latitude	Freezing Rain (mm)	Precipitation (mm)
Charlo A	-66.33	47.98	0.0	Missing
Bathurst A	-65.75	47.63	2.8	63.2
St Leonard A	-67.83	47.15	0.0	53.8
Miramichi A	-65.47	47.01	10.6	82.9
Kouchibouguac	-65.00	46.77	20.0	107.7
Bouctouche CDA	-64.77	46.43	26.0	40.2
Charlottetown A	-63.13	46.29	16.8	45.5
Sydney A	-60.05	46.17	0.0	27.4
Moncton A	-64.69	46.10	31.2	78.4
Fredericton A	-66.53	45.87	9.0	39.1
Amherst	-64.27	45.85	9.0	44.2
Gagetown AWOS	-66.43	45.83	35.0	41.0
Sussex	-65.53	45.72	30.6	77.2
Wolfe Lake	-65.15	45.67	58.3	68.1
FP Alma	-64.95	45.60	2.2	55.2
Saint John A	-65.89	45.32	9.7	59.1
St Stephen	-67.25	45.22	6.6	47.2
Point Lepreau CS	-66.45	45.07	9.7	59.1
Greenwood A	-64.92	44.98	6.0	74.4
Grand Manan	-66.80	44.72	0.0	25.7
Shearwater A	-63.50	44.63	0.0	61.6
Yarmouth A	-66.08	43.83	0.0	42.0

Table 5-2. Precipitation Amounts, Feb 2-3, 2003 (ref. Richards)

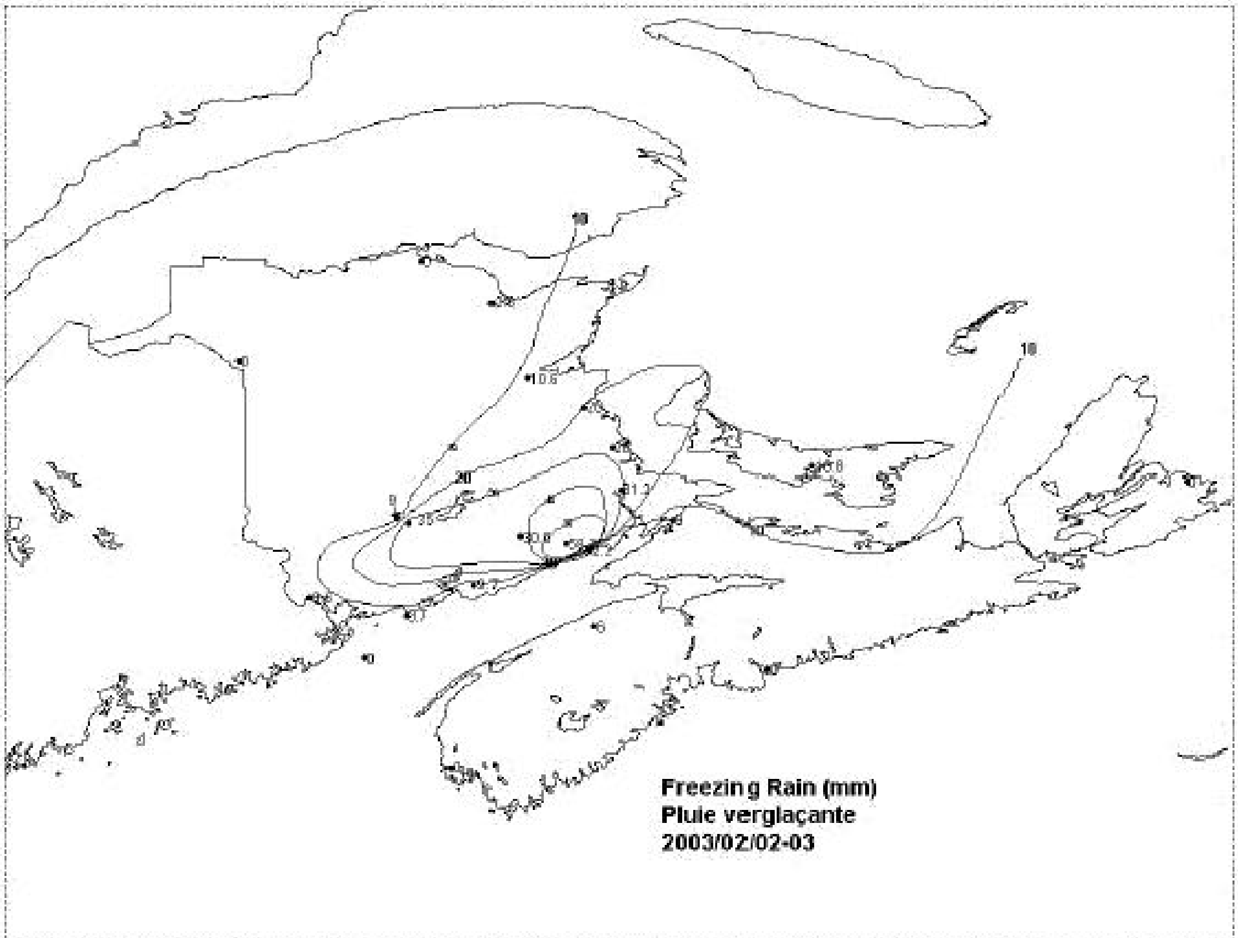

Figure 5-14. Freezing Rain Amounts (mm), Feb 2-3, 2003 (ref. Richards)

Figure 5-15 shows an extreme value analysis of freezing ice thickness, using the data from the Saint John weather station, 1953 to 2003.

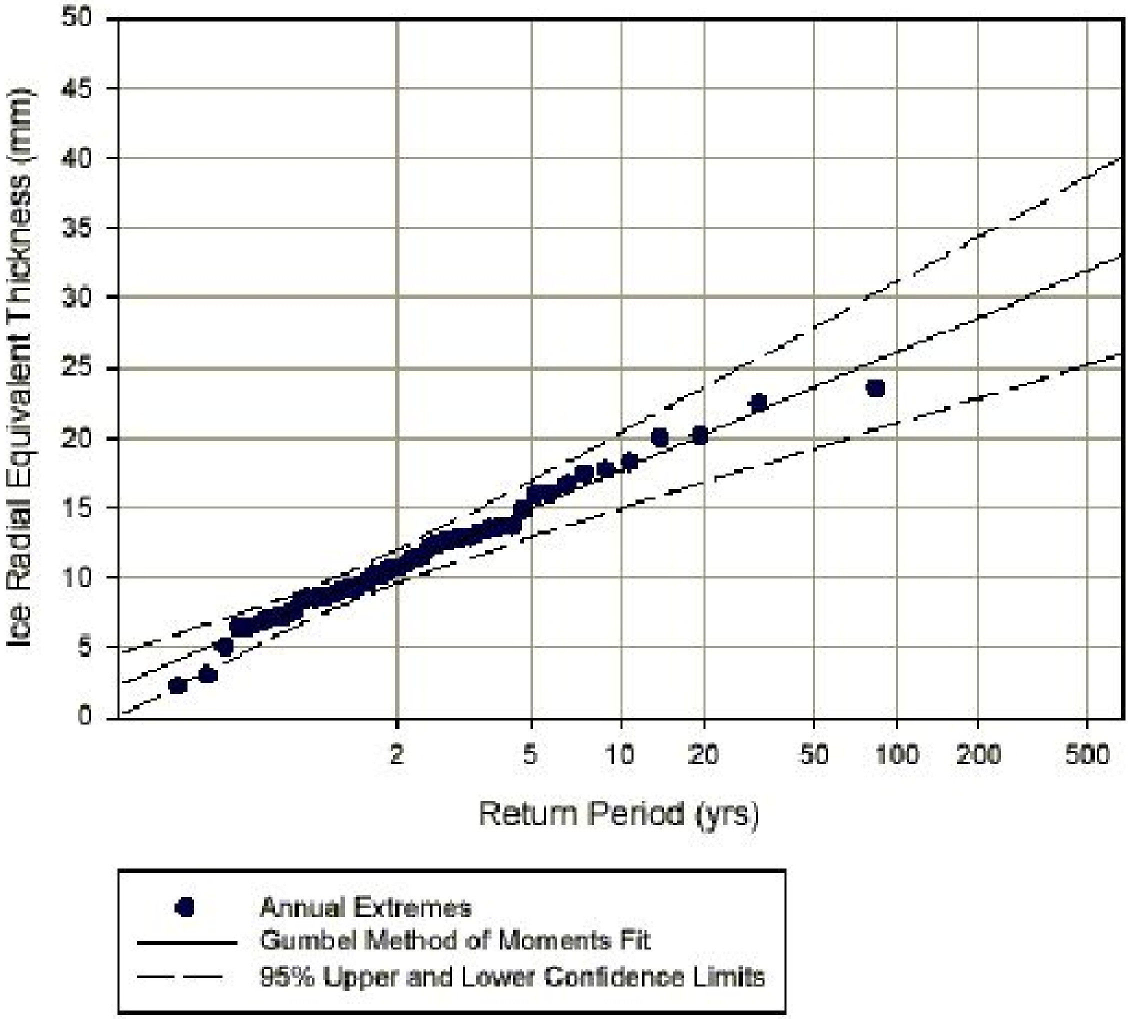

Figure 5-15. Saint John Airport, Ice Radial Equivalent Thickness, Extreme Value Analysis, 1953-2003 (ref. Richards)

5.4 Reliability Model

In Section 3.3, we described how to set the strength capacity for three load cases in common use by BPA for a four-legged steel lattice transmission tower: transverse extreme wind; rime ice + transverse concurrent wind; or glaze ice + transverse concurrent wind. The underlying assumption in these models is that failure is initiated due to excessive transverse forces on the tower; other failure modes (including unbalanced ice loading, insulator failures, connection hardware failures, conductor failures, etc.) are not considered controlling in the extreme wind models used in Sections 3.3 and 3.4.

In the failure of Line 3002, the loading conditions may be somewhat different as those for the BPA extreme load cases. The nature of the Line 3002 initiation failure at tower 294 suggests the following:

- The southeast guy wire on structure 294 failed. This guy wire would be loaded in tension due to unbalanced dead weight (and ice weight) loads (span north of 294 is longer than span south of 294); as well as transverse winds from the NNE.

- Failure of the guy wire might have been caused by unbalanced ice loading (dead weight only), given the long horizontal span on the north side of 294 (487.7 meters) and the short horizontal span on the south side of 294 (260.1 meters); coupled with the change in elevation between 294 and 293 (the conductor attachments on 293 are about 115 feet lower than on 294); coupled with the change in exposure of the line as it goes over the local hillcrest at 294 (perhaps more early sun exposure between 294-295 than between 294-293). (Data from Figures 5-2, 5-6, Table 5-2). The unbalanced loads on the shield wires as well as the conductors would result in high tension loads on the southeast and northeast guy wires for tower 294.

- Failure of the guy wire might have been caused by unbalanced (or balanced) ice loading plus concurrent wind from the NNE. The wind at the time of tower failure is estimated as ~8 to ~10 mph at weather station CYSJ (8 mph if one assumes the wind at CYSJ is duplicated at tower 294 about 2 hours later; 10 mph if the wind at CYSJ is assumed to be the same at tower 294). The transmission line direction between towers 293 and 295 is nearly perpendicular to the prevailing wind direction (from the NNE) at the time of failure. Tower 294 is situated atop a ridge of a hill, which may have contributed some modest wind speed-up due to topographic effects. The actual wind loads would have created tension on the southwest and southeast guy wires.

- The net effect of the ice loads and wind loads would have created the highest tension in the southeast guy wire. Since the southeast guy wire actually broke, a reasonable assumption is that there was ice with some concurrent wind from the NNE at the time of the collapse.

Unlike the three load cases of Section 3.3, the actual failure mechanism appears to have been a high longitudinal load on tower 294, as the southeast guy wire snapped; and this initiated the collapse and subsequent cascading.

Campbell reports that the actual guy wire failed at 75% of rated strength, assuming 1" of radial ice with no concurrent wind, and assuming the site-specific horizontal span of 370 m (1215 feet). Let us also assume that the towers were designed for a 100 mph transverse wind, assuming a 1500 foot span.

Since the true failure likely had some contribution from both ice and wind, we can set up three constraints for the design strength criterion, assuming the design span of 1,500 feet:

- When the actual radial ice thickness is 1.08 inches, (= 1"/0.75 * (1215/1500')) the tower reaches its nominal strength limit.
- When the actual radial ice is 0.75" and there is a concurrent wind speed of 56 mph.
- When the wind speed, with no ice, is 100 mph.

Figure 5-16 shows this two-dimensional strength criterion. Outside of the "Design Strength Boundary", the tower becomes increasingly unreliable. At the "Design Strength Boundary", the tower should be reasonably reliable (perhaps 2% to 15% chance of failure). Well within the "Design Strength Boundary", the tower should be extremely reliable (essentially no chance of failure).

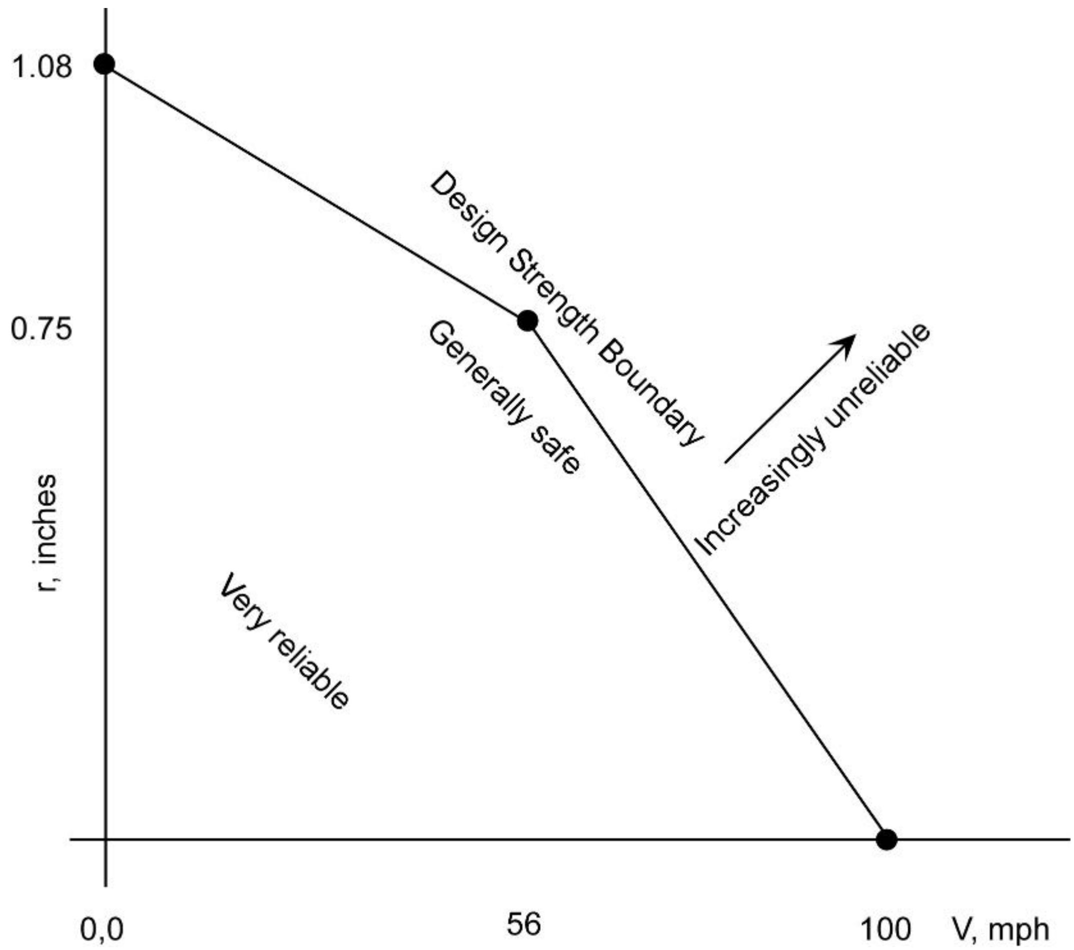

Figure 5-16. Strength Criterion

Full scale tests of towers suggest that about 50% of them collapse when V(actual) / V(design) is about 1.12. So, to develop a failure boundary, V(failure) = 112 mph in this case.

Let us assume that the failure boundary (50% of the time) for the case of ice only loading is 25% higher than r needed to reach its strength limit, or r(failure) = 1.08 * 1.25 = 1.35 inches.

Since failures based on r (only) are load-based, and failures based on V (only) are velocity based, it makes sense that a combined failure criterion should vary linear with r, but as the square of V. Conceptually:

$$fragility_{wind+ice} = \left(\frac{r_{actual}}{r_{fragility}}\right) + \left(\frac{V_{actual}}{V_{fragility}}\right)^2$$

For Level 1 and 2 models, we have tributary horizontal span lengths versus design horizontal span lengths that can be used to vary the results. While for ice loading, it would be best to use unbalanced loading for the ice-only condition, this would require knowledge of vertical profiles, sags, line tensions; but this information is not assumed to be available except for Level 3 analyses. To include horizontal spans, the simplified failure criterion for combined loading becomes:

$$failure_{wind+ice} = \left(\frac{L_{actual}}{L_{design}}\right)\left[\left(\frac{r_{actual}}{r_{fragility}}\right) + \left(\frac{V_{actual}}{V_{fragility}}\right)^2\right]$$

Table 5-3 lists the criterion for several various assumptions for r(design) and V(design), Beta, L(design), L(actual) and actual ice and wind conditions.

Case	r Design (inches)	r Actual (inches)	V Design (mph)	V Actual (mph)	L Design (feet)	L Actual (feet)	Failure Criterion	Beta Total	P(fail)
1	1.08	1.35	100	0	1500	1500	1.0	0.3	0.500
2	1.08	0.00	100	112	1500	1500	1.0	0.3	0.500
3	1.08	0.75	100	56.6	1500	1500	0.81	0.3	0.242
4	1.08	0.50	100	56.6	1500	1500	0.63	0.3	0.059
5	1.08	1.00	100	15	1500	1215	0.61	0.3	0.052
6	1.08	1.00	100	0	1500	1215	0.60	0.3	0.044
7	1.08	1.00	100	30	1500	1215	0.66	0.3	0.081

Table 5-3. Tower Reliability Criterion – Type A, B Tower

The following discusses the findings in Table 5-3:

- Case 1. This represents an extreme ice load with no wind, on a full-span tower (vertical axis in Figure 5-16). As expected, the chance of failure is 50%.
- Case 2. This represents an extreme wind load with no ice, on a full-span tower (horizontal axis in Figure 5-16). As expected, the chance of failure is 50%
- Case 3. This represents a design-basis ice with concurrent wind, on a full-span tower (56.6 mph/0.75" combination in Figure 5-16). The chance of failure is 24%. This is within reason as to the true failure probability.
- Case 4. This represents a design-basis ice with concurrent wind, on a full-span tower (56.6 mph/0.50" combination). The chance of failure is 5.9%. This is within reason.
- Case 5. This represents an estimate of the upper bound of the actual loading condition on April 3, 1993 (1 inch ice, per Campbell, plus 15 mph concurrent wind if topographic effects are included). The chance of failure is 5.2%. This seems somewhat high, as we observed only 1 failure out of a population of perhaps 50 or more similar towers exposed to this loading.
- Case 6. This represents a best estimate of the actual loading condition on April 3, 1993 (1 inch ice, per Campbell, plus 0 mph concurrent wind). The chance of failure is 4.4%. This seems somewhat high, as we observed only 1 failure out of a population of perhaps 50 or more similar towers exposed to this loading.

Based on the results in Table 5-3, we believe that likely hypotheses for the initial failure include:

- That the southeast guy wire splice was perhaps installed incorrectly for Tower 294, resulting in a tower-specific weakness; or
- That the actual ice loading was more than 1 inch; or
- That the effects of unbalanced ice loading, possibly with some cable dynamics as some ice fell off the 294-295 span, led to the high load on the southeast guy wire; or
- Case 7. That a higher gust of wind occurred at tower 294 (say about 30 mph), which leads to a 8.1% chance of failure for Tower 294.

Of all these hypotheses, we suggest that Tower 294 was installed / constructed / aged to have a defective guy wire; or a more complex cable dynamics situation arose due to unique unbalanced loading. It would require Level 3-type refinements to further explore the issue.

Given the above, we modified the wind-only models described in Sections 3 and 4 to include ice loading for Line 3002. The ice plus concurrent wind loading model is included in file *Line 3002 0.xls*. The column AG "Failure Criterion" in the ice loading model incorporates both the ice and wind loading parameters given above, adjusted to accommodate variations in horizontal span lengths from design spans, and user-entered Kzt (topography), tower-specific local over-strength (or under-strength).

We used this model to examine several parametric studies.

Case	Level	Wind Speed MPH (294/295, range)	Ice Inch (294/295, range)	Beta Total	Tower 294 Local Strength	D/C Cutoff	Tower Failures (initiations)	Towers with p(fail) > 1%	Towers with p(fail) > 5%	Tower 294 p(fail)
201	1	15 - 5	1.00 – 0.00	0.30	1.00	0.50	1.41	46	7	0.029
202	1	14, 10 – 5	1.00 – 0.00	0.30	1.00	0.50	1.28	45	5	0.027
203	1	14, 10 – 5	1.00 – 0.00	0.30	0.75	0.50	1.41	45	6	0.165
204	1	14, 10 – 5	1.00 – 0.00	0.30	0.75	0.60	0.63	9	6	0.165
205	1	14, 10 – 5	1.00, 1 - 0	0.30	0.75	0.60	1.04	12	12	0.237

Table 5-4. Parameter Study, Line 3002

Case 201. For Case 201, we assume that the wind along Line 3002 varied from 15 mph (southern end of line at coast) to 5 mph (northern terminus of line). This is clearly a bit simplistic, as we only have weather station anemometer data at the Saint Johns airport suggesting perhaps 8 to 10 mph at some point inland form the Bay of Fundy.

We assume that the ice along Line 3002 varied from 0.00 inches (northern end of line, inland) to 1.00 inches (southern terminus of line). In the vicinity of the tower failures, the ice thickness is assumed to vary from 0.92" to 0.96". This is clearly a bit simplistic, as we only have observations of "about" 1 inch of radial ice in the vicinity of Tower 294.

We assume a "design" strength of 100 mph wind (no ice) or 1.08 inch radial glaze ice (no wind), and all towers have a design span of 1,500 feet. As we have no original design data for any of the towers, these assumptions are based on judgment that the actual maximum spans along the alignment are

1542 feet and average are 1,214 feet for tangent towers, and the actual maximum span along the alignment is 1346 feet and average is 1,123 feet for angle towers.

Results. Case 201 shows a total number of tower failure initiations of 1.41. 46 towers have more than a 1% chance of failure, 7 tower have more than a 5% chance of failure, and tower 294 has a 2.9% chance of failure. The maximum chance of failure of any tower along the alignment is 9.0% (Tower 293).

Case 202. In Case 202, we begin with Case 201, and then we reduce the average wind speeds to range from 10 mph (coast) to 5 mph (inland). For towers 294 and 293 specifically, as they are on hills, use Kzt = 1.5 to increase their wind speed to about 14.5 mph.

Case 203. In Case 203, we begin with Case 202, and then we assume a strength defect on Tower 294 guy wire splice, such that the splice is 75% of the rated guy wire capacity. This increases the chance of failure for Tower 294 from 2.7% to 16.5%.

Case 204. In Case 204, we begin with Case 203, and then increase the cut-off from D/C of 0.50 to 0.60. This reduces the total number of tower failure initiations to 0.63.

Case 205. In Case 205, we begin with Case 204, and then set the actual ice loading for all towers from 291 southwards to the coast at 1.00 inches. This increases the total number of tower failure initiations to 1.04, and the chance of failure of Tower 294 at 24%.

The cascading / zippering of 14 adjacent towers after Tower 294 collapsed is very important to overall line reliability, as well as making decisions as to how strong to design each tower both for external loads and unbalanced line loads. This important aspect for reliability analyses is described in Section 6.

6.0 CASCADING EVENTS

In the three examples presented in Section 3, 4 and 5, we considered *only* the failure of towers due to the initial overloads due to external wind or ice loads. In Section 6, we present a model that can be used to examine the cascading effects on the transmission circuit as a whole, given the probability of initiation failure for each tower along the circuit.

To examine this problem, let us consider a hypothetical circuit that includes 11 towers: a dead end tower at each end, an angle tower in the middle, and 4 light suspension towers between the middle tower and each dead end tower. The example is illustrated in Figure 6-1.

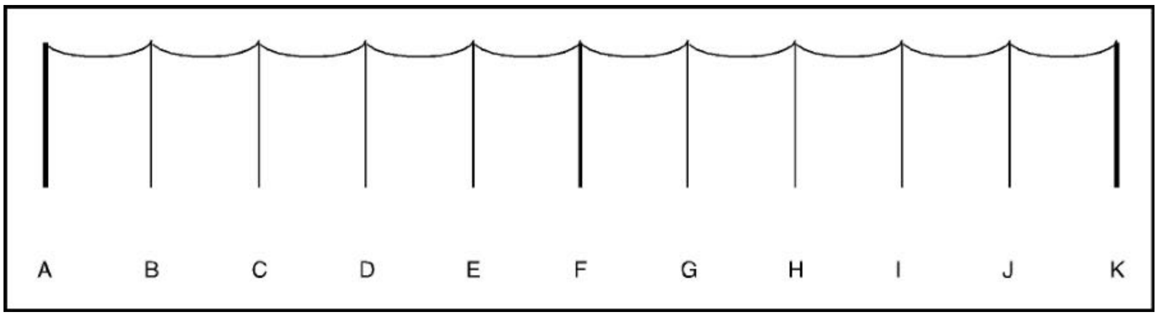

Figure 6-1. Example Circuit

Table 6-1 provides the assumed properties for each of the towers. The two key properties of interest are as follows:

- Given an extreme storm (wind or ice) event, what is the probability that the tower collapses and drops all phases to the ground? This data would normally be taken from the computations described in Sections 3, 4 or 5 of this report. For purposes of the example in Section 6, this data is assumed, and listed in column 3 of Table 6-1.
- Given that the tower survives the external loading from the extreme event, what is the probability that the tower collapses due to zippering (i.e., the tower collapses as all phases on one side of the tower are collapsed, and the phases on the other side are still up?)

Tower	Type	(Collapse due to external wind / ice load)	(Collapse due to collapse of one adjacent tower)
A	Dead End	0.01	0.001
B	Suspension	0.2	0.9
C	Suspension	0.2	0.9
D	Suspension	0.2	0.9
E	Suspension	0.2	0.9
F	Angle	0.2	0.5
G	Suspension	0.2	0.9
H	Suspension	0.2	0.9
I	Suspension	0.2	0.9
J	Suspension	0.2	0.9
K	Dead End	0.01	0.001

Table 6-1. Example Circuit

Two failure probabilities are provided in Table 6-1 for each tower. The first probability reflects the chance that the tower fails on its own (initiation failure) due to the externally-applied extreme load. The second probability reflects the chance the tower fails if it is exposed to a complete unbalanced load (zippering, all phases intact on one side, all phases failed on the other side).

For this example, we selected the probabilities to reflect a circuit that starts and ends at two substations. With only 11 towers, this is a rather short circuit, but adequate for demonstration purposes. The following outlines the sense of the assumed failure probabilities.

- External load failure probability (). At each substation, there is a dead end tower that has been designed for wind and ice loads with a large factor of safety. It is assigned a chance of failure of 0.01 given the occurrence of an extreme loading event. The remaining towers are designed to meet the minimums of ASCE 74 (or similar), and so are assigned a 0.2 chance of collapse given the extreme event. In other words, the external loads are perhaps in the range of about 90% to 105% of the design load.

- Zippering failure probability (). At each substation, there is a dead end tower that has been designed for full unbalanced loads, and thus the chance of zippering is set rather small (1 in 1,000). In the middle of the circuit is an angle tower; while this tower might not have been explicitly designed for full loss of all phases, but owing to the geometry of the angle tower, it has a fair amount of resistance for unbalanced loading anyways, so it is assigned a chance of failure of 0.5 (perhaps this might be in the range of the capacity of angle structure 295 in Section 5). The suspension towers are assumed to have not been designed for full unbalanced loads, and are assumed to have a high chance of failure (0.9) if the adjacent tower collapses.

- The chance of initiation failure does not spatially vary in this example. In the real world, there will be spatial variation as the external wind (ice) loading will vary over distance and in consideration of local topographic conditions.

The computation of is outlined in Sections 3, 4 and 5. The computation of is not addressed in detail in this report; sample values provided in this report are based on the judgment of the authors, coupled with some limited studies done at BPA and elsewhere. For some guidance on , including possible mitigation strategies, see Section 3.3, Longitudinal Loads, ASCE 74 (2008). The emphasis below is the mathematics used to develop the total failure probabilities of each tower, as well as the reliability of the circuit as a whole.

The model is presented in Table 6-2, with the computation model provided in file Cascade 1.xls under the sheet "Model". The explanation of each column follows.

Tower (A)	Chance of Collapse due to Direct Extreme Wind/Ice Load (Pe) (B)	Chance of Zipper if Adjacent Tower Collapses (Pz) (C)	Number of Zippered Towers if Tower Collapses (D)	Total Tower Failures if this tower has an initiating collapse (E)	Given extreme event, number of collapsed towers (F)	Given Extreme Event, Chance this tower Fails (G)	Given Extreme Event, Chance this tower Fails, LIMIT 1 (H)	Limiting chance this tower will fail (I)	MIN (H, I) (J)
A (Dead End)	0.01	0.001	4.439	4.449	0.054	0.011	0.011	0.011	0.011
B	0.2	0.9	3.933	4.133	0.987	1.056	1.000	2	1
C	0.2	0.9	4.270	4.470	1.054	1.129	1.000	2	1
D	0.2	0.9	4.454	4.654	1.091	1.173	1.000	2	1
E	0.2	0.9	4.488	4.688	1.098	1.187	1.000	2	1
F (Angle)	0.2	0.5	6.192	6.392	1.438	0.894	0.894	1.2	0.894
G	0.2	0.9	4.488	4.688	1.098	1.187	1.000	2	1
H	0.2	0.9	4.454	4.654	1.091	1.173	1.000	2	1
I	0.2	0.9	4.270	4.470	1.054	1.129	1.000	2	1
J	0.2	0.9	3.933	4.133	0.987	1.056	1.000	2	1
K (Dead End)	0.01	0.001	4.439	4.449	0.054	0.011	0.011	0.011	0.011
Total	1.82	7.702			10.005	10.005	8.916	17.222	8.916

Table 6-2. Cascade Model

Column A lists the name of the tower, with a brief description. User entered.

Column B lists the failure probability of each tower due to external loads (). User entered in this example, but computed using reliability techniques described in Sections 3, 4 or 5 this report. The Total row at the bottom of the table is the simple addition of the 11 values in column B. In other words, we expect 1.82 towers to collapse due to external loads. This value of 1.82 failed towers assumes no zippering phenomena.

Column C lists the failure probability of each tower due to zippering (P_z). User entered in this example. The selection of P_z is based on a combination of expert judgment, empirical observations, and limited dynamic nonlinear analyses. The Total row at the bottom of the table is the simple addition of the 11 values in column C. This Total value of 7.702 is an upper bound of the number of towers that can fail due to zippering, but ignores that zippering of some towers might not occur as there is some chance that both adjacent towers will not collapse either due to external loads or due to zippering.

Column D lists the total number of adjacent towers that collapse due to zippering should the tower on that row collapse due to external loads. The formula for Tower A is:

$$N_z^A = P_z^B + P_z^B * P_z^C + P_z^B * P_z^C * P_z^D + \ldots = \prod_{i=B}^{i=K} P_z^i$$

- The first term on the right side of the equation (P_z^B) is the chance that tower C collapses due to zippering (90%) given that Tower A collapses due to external loads.
- The second term on the right side of the equation ($P_z^B * P_z^C$) is the chance that tower D collapses due to zippering (81% = 90% * 90%) given that Tower A collapses due to external loads.
- The third through last terms are the chances that subsequent towers collapse due to zippering, given that Tower A collapses due to external loads. For this example, there are B through K towers (10) that are checked for zippering failures due to the collapse of Tower A.
- In the equation above, this is represented by $\prod_{i=B}^{i=K} P_z^i$, which is the multiplicative sum of the zippering probabilities of each tower in the chain.
- For a collapse of Tower A, the total number of collapsed adjacent towers due to zippering is 4.439.
- For a collapse of Tower B, the computation is similar, except that the "chain" of multiplications proceeds up the table as well as down the table.
- For a collapse of Tower K, the computation is similar, except that the "chain" of multiplications proceeds up the table.
- This example shows that the most overall damage occurs if Tower F collapses (6.192), as this exposes the two longest chains exposed to zippering. This example shows that the least overall damage occurs if either Towers B or J collapse (3.933), as the chain of collapsed zippered towers is largely eliminated on one direction by the very robust dead end towers A or K.
- While column D provides some information as to the overall number of collapsed towers, it does not provide information that can be used to estimate the overall circuit reliability.

- The number of computations increases geometrically as the number of towers increases. For the example, the number of multiplications is about (ignoring the bifurcated chain) 1 + 2 + 3 + 4 + 5 + 6 + 7 + 8 + 9 = 45, and the number of additions is 9. Allowing that a multiplication takes about 10 times as much CPU time as an addition, the computation effort is about 11 * (45*10 + 9) = 5049. If the number of towers in the circuit is increased from 11 (this example) to 22, the compute effort increases from 5049 to 46,640, or a nine-fold increase. If the number of towers in the circuit is increase from 11 (this example) to 110, the compute effort increases from 5049 to 6,486,480, or a 5,049-fold increase. A circuit with 300 towers would have a compute effort of about 133 million. While a modern desktop or laptop computer is "fast", we can expect some real-clock delay in doing the entire chain of computations for a long circuit.

Column E is the total number of towers collapsed, assuming the initiating tower (A) collapses and including the zippered towers (B through K), or 0.01 + 4.439 = 4.449.

Column F is the number of towers collapsed given occurrence of the extreme event. It is computed as:

$$N^A = P_e^A + P_e^A * N_z^A$$

- For Tower A, this is 0.01 + 0.01 (4.439) = 0.054.
- For all Towers A through K, the total is 10.005 out of 11 towers.
- This computation, while simple to compute, results in an overestimation of the total number of collapsed towers on the circuit. This is because it assumes that tower failures due to zippering are independent; but, in fact, this double-counts as a single tower cannot have more than 100% chance of collapse, either due to initiation or zippering.

Column G re-formulates the computation of Column F, in order to get a better estimate of the chance that any tower will collapse, given the extreme event.:

$$N^A = P_e^A + P_e^B * P_z^A + P_e^C * P_z^B * P_z^A + \cdots + P_e^K \prod_{i=B}^{K} P_z^i$$

- The total in Column G (10.005) is the same as the total in Column F (10.005), but the "physical" interpretation of the computations in column G is closer to physical reality. For example, the probability of collapse of tower A is computed as 0.0110468 (to 7 decimal places), which is very near the practical limit of 0.01 (initiation) plus worst case zippering (0.001) = 0.011 (exact).
- However, the Column G computation still allows a greater than 100% chance that a tower fails, which is physically incorrect. This occurs as Column G is the summation of independent zippering calculations, each of which assumes that no other tower has collapsed already. In other words, Column G has some double counting. When the probability of collapse and zippering values are small (like for Tower A), the error in double counting is miniscule (0.0110468 versus exact value of 0.011), but when the values are large, we get nonsense results like Tower E having a 118.7% chance of collapse (clearly at least 18.7% too high).

Column H presents a simple limit function to prevent the over (double) counting problems in Column G:

$$N^{A} = MIN\left[N^{A}_{\text{Column G}}, 1\right]$$

- By applying this limiting function, we get a total number of tower collapses of 8.916. This is fewer than the 10.005 value by either Columns F or G.
- Column H still suffers from the slight error in that Tower A still has a chance of failure of 0.011048, which is higher than its theoretical limit of 0.011.
- The total number of Tower failures is 8.916455.

Column I lists the theoretical limit of failure for each tower:

$$P^{B} = P^{B}_{e} + 2 * P^{B}_{z}$$

- Column I sets the maximum chance that a tower will fail as the chance it will fail due to external loads, plus 2 times that chance it will zipper if either of the two adjacent towers fail. Note, for Towers A and K (first and last), the "2" value is replaced with "1", as there is only 1 adjacent tower for these dead end towers (this reasonably ignores that chance of collapse of the dead-end tower initiated by collapses of equipment within the substation).

Column J is the final result, and minimizes the chance of tower collapse from columns H and I.

$$N^{A} = MIN\left[N^{A}_{\text{Column H}}, N^{A}_{\text{Column I}}\right]$$

- The total number of Tower failures is 8.916361 = $T_c = \sum_{i=A}^{K} N^{i}_{\text{Column J}}$
- While the result in Column J is very nearly the same as Column H, the extra computational effort for Column J is trivial.

The reader is cautioned that the accuracy of these results, using the above example, is provided only to help understand the computational process. In practice, it is not likely that the calculations will be more accurate than 1 significant digit.

Using the information in Column J, we can compute the circuit reliability. We define this to mean the chance that the circuit will be functionally undamaged, given the extreme event. Two approaches are considered:

$R_1 = 1 - T_c, R \geq 0.0$
or
$R_2 = e^{-T_c}$

where Tc is the total number of tower failures along the circuit.

- In the first model (R_1), if the total number of damaged towers, T_c is 0.6, then there is a 0.4 chance that the circuit will be reliable.
- In the second model (R_2), if the total number of damaged towers, is 0.6, then there is a 0.54881 chance that the circuit will be reliable (0.54881 = exp(-0.6)).
- For small values of T_c (say 0.2 or less), the two reliability models are very nearly the same. For larger values of T_c (say 0.5 or higher), the two reliability models begin to diverge. The following lists the differences.
  - Tc = 0.01 R1 = 0.99 R2 = 0.99005
  - Tc = 0.1. R1 = 0.90 R2 = 0.90484
  - Tc = 0.2 R1 = 0.80 R2 = 0.81873
  - Tc = 0.5 R1 = 0.50 R2 = 0.60653
  - Tc = 0.6 R1 = 0.40 R2 = 0.54881
  - Tc = 0.9 R1 = 0.10 R2 = 0.40657
  - Tc = 2 R1 = 0.00 R2 = 0.13534
  - Tc = 5 R1 = 0.00 R2 = 0.00674
- For purposes of this report, either measure of reliability has some merit. The first model is very simple to understand, and shows that once Tc is 1 or larger, the circuit is entirely unreliable. The second model assumes that a circuit with more than one collapsed tower still have some small chance of being able to transmit electricity.
- Which model to use depends on what exactly is meant by "collapse" of a tower. If the "collapse" means that one or more conductors has dropped so low to the ground as to fault, then model 2 has no merit. If, on the other hand, a "collapsed" tower might still have a chance to be standing (albeit with some gross deformation) and keeping the distances between each phase of the conductors and the tower and the ground above the arc-distance, then at least some "collapses" might not be so severe as to fault the circuit.
- A circuit with reliability of much under 80% is not going to be readily acceptable from the owner's point of view, unless other undamaged circuits of the network as a whole can sustain the system. Most transmission systems have "N-1" designs, and even a few have "N-2", so in these cases, having R of about 80% for one circuit might be acceptable, as long as all the other circuits have R nearly equal to 1.0.

For purposes of the rest of this report, we use the R2 definition for reliability, which has the benefit of being a smoother model than R1, and perhaps a little better in corresponding to some chance that a severely damaged tower might not actually immediately fault the circuit; but would still require post-storm repair.

7.0 BENEFIT COST ANALSYIS

In Sections 3 through 6, the transmission tower reliability analysis is performed for a *scenario* storm. A *scenario* storm is a single storm event that might take place over a few hours to a few days. A scenario storm has the following characteristics:

- A particular wind speed (or ice thickness with coincident wind speed) at each tower location.
- If Level 3 analyses are performed, the wind velocity might be incorporated (i.e., the direction of the wind, as it varies with time over the duration of the storm).

Analyses using a scenario storm has the following benefits:

- Results are based on a single storm. This allows the utility to understand the post-storm emergency response needs, by adding up all the damage to all the circuits and towers in its entire system.
- Results can be calibrated to known historical storms. Calibration can be done to reflect local variation of wind speeds, individual tower fragilities, variations in dispersions, etc. It should be relatively straightforward to calibrate the model to be able to accurately forecast tower damage.

If the user does not specify the recurrence interval of the scenario storm, the results do not provide enough information as to make informed decisions as to whether or not it is worthwhile to invest more capital money into the circuit to strengthen towers (or conversely, save capital costs and build weaker towers). In order to make this type of important decision, one *should* consider the frequency of storms, as well as a few other factors:

- Annual chance of various size storms. For purposes of analysis, the example in this section assumes the return periods are 10 years, 20 years, 50 years, 100 years, 200 years, 500 years, 1,000 years, 2,500 years and 10,000 years. While fewer or more than nine intervals can be used, the example model uses these nine return periods. The 10- and 20-year return periods are especially important to understand if the circuit being examined has been subject to repeated damage over short intervals; if so, upgrade will almost certainly be warranted. The 50-year return period is selected as it is commonly the value in most codes. The 100-year to 1,000 year return periods attempt to capture fairly rare storms. The 2,500 to 10,000 year return periods are selected to capture what might be the worst storm (like a direct hit by a category 5 hurricane in Florida) or even tornados.
- Annual losses due to each of the nine storms (10, 20, 50, 100, 200, 500, 1000, 2500, 10000 years).

- Mitigation effectiveness. The user must assume a mitigation scheme (say strengthen select towers), and re-run the analysis for the "upgraded" circuit to establish its outcomes in each of the five storms.

- Economic impacts. The decision to upgrade a circuit will depend not only on the losses due to direct damage of the towers, but also the economic impacts to the served community (rate-payers) should there be damage to a circuit.

- Capital and ongoing maintenance costs.

- Economic lifetime.

- Discount rate.

- Net Present Value (NPV)

- Benefit Cost Ratio (BCR)

7.1 Overview of Benefit Cost Analysis

A benefit is the *reduction in future losses* should a particular mitigation action be taken. To compute the benefit, the reliability analysis must be run twice: once for the as-is system, and once for the hypothetically-upgraded system. The projected losses are computed for each system. The benefits are the losses (as-is) less the losses (mitigated).

$$BCR = \frac{\sum_{i=1}^{n} \frac{Benefit_i}{(1+r)^i}}{Cost_0}$$

Where $\boldsymbol{Benefit_i}$ is the benefit in year i from performing the mitigation, r = discount rate, $\boldsymbol{Cost_0}$ is the capital cost in year 0. The analysis is performed over n years, where n should normally be taken as 50 to 100 years or so.

All costs and benefits are presented in current year dollars, and *not* in future inflated dollars. This simplifies the analysis by not having the user decide what the forecast rate of inflation might be. The discount rate r is for the *real* time value of money. If the user wanted to use future inflated dollars, then the benefits must be inflated for inflation (say at rate i) and the discount rate would have to be increased to (r+i) to reflect the time value of money that includes inflation. As the inflation value i is in both the numerator and denominator, the inflation effect essentially cancels out (at practical levels of inflation) and there is no change in the analysis.

The value n is important for doing the BCA, but *does not* reflect the true lifetime of transmission towers that might have very long functional lives. With proper ongoing maintenance, the actual life of a steel lattice tower might be several hundred or even thousands of years (for wood poles, it might be under 75 years). When doing the BCA, selecting n greater than 100 years usually makes very little difference in the calculation, if r is selected at a suitably high value like 7%.

For analysis of single circuits in networks with N-1 reliability, the BCA can reasonably make the simplifying assumption that benefits are simply the reduction in future losses to the transmission tower owner due to direct damage of the tower itself.

For storms that affect more than 2 circuits, or for a circuit with only N reliability, it is incorrect to only consider repair and re-construction costs for purposes of deciding what level of mitigation is most suitable, as it excludes important benefits that include:

- Loss of revenue to the owner should the tower be damaged (if the owner cannot sell power while the circuit is faulted).
- Loss of economic activity to the rate-payer should the circuit be faulted. This is likely a value on the order of, or much higher than the direct damage costs.
- Injuries or fatalities due to collapse of the tower.
- Detour impacts due to damage of the tower, including disruption to road traffic (detour time) or rivers (delays of water shipping) due to excessively sagged / fallen conductors. Should these factors be incorporated, one will likely conclude that towers either side of rivers or major highways should be designed for a higher reliability.
- Other secondary impacts, including potential ignitions and fires, impact of towers on adjacent structures or facilities, etc.

All of the above should be considered direct economic losses, as they accumulate to the community served.

Indirect economic losses / impacts might include *increases* in economic activity outside the service area, due to damage within the service area. For example, should a tower collapse result in an extended power outage to a specific community, a shopper in that community might travel to another community outside the service area. So, for example, a supermarket with the affected area loses economic activity (a direct economic loss) but a supermarket outside the affected area gains economic activity (an indirect economic gain). Such indirect economic losses (gains) are not usually accounted for in BCA, as they accumulate to non-rate payers (i.e. customers located outside of the area served by the utility)

7.2 Benefit Cost Analysis, Suspension Tower Upgrade

We introduce the Benefit Cost Analysis (BCA) by using two examples. Example 1 is the model described in Section 6, with suspension and angle towers designed for 100 mph, and with suspension towers having very low capacity to withstand zippering (10%) and angle towers modest capacity (50%).

Example 2 assumes that the suspension and angle towers are upgraded to have a robust capability to withstand zippering (99%). The complete computations for these two examples are in the Excel file *Cascade 1.xls*, sheets *Ex 1* and *Ex 2* (contact G&E if you are interested in the Excel files).

Table 7-1 lists the results for a range of assumptions. Example 1 is the same as the model listed in Table 6-2, with two dead end (DE) towers (very robust), 1 angle (A) tower (modest strength, modest zippering capacity), and 8 suspension (S) towers (modest strength, little zippering capacity).

Example	(DE, S, A) Column B	(DE, S, A) Column C	N Column B	N Column J	Circuit Reliability R2	Repair Cost Column L
1	0.01, 0.2, 0.2	0.001, 0.9, 0.5	1.82	8.916	0.000134	$938,554
2	0.01, 0.2, 0.2	0.001, 0.01, 0.01	1.82	1.853	0.157	$289,155

Table 7-1. Benefit Cost Examples (ref. *Cascade 1.xls)*

Example 1 is the same as used in the model, Table 6-2. This is a circuit with many weak suspension towers having little longitudinal capacity to withstand zippering. The external storm loads are on the order of the design basis used for initial tower construction. Most of this circuit is expected to collapse. In Example 1, the suspension towers might be built with guyed steel lattice towers, using (relatively) low cost guy wires for longitudinal resistance.

Example 2 has the same collapse probabilities due to external loads, but assumes that all non-Dead End towers have very robust capability to withstand full unbalanced loads. We see that the total number of tower collapses is reduced from 8.916 to 1.853 (79% reduction in damage), but circuit reliability is only increased from 0.01% to 15.7%. Example 2 might be the case that the suspension towers are made from 4-legged steel lattice towers, with each tower being designed to resist full unbalanced line loads.

The question is: is the added expense for the tower design in Example 2, worth it as compared to Example 1? Let us examine this situation, with the following assumptions:

- A guy-wire suspension tower not designed for full unbalanced load costs $100,000 to install.
- A dead end tower costs $200,000.
- An angle tower with modest zippering capacity costs $150,000. An angle tower with complete zippering capacity, is effectively a dead-end tower, and costs $200,000.
- A 4-legged lattice tower designed for full unbalanced load costs $150,000 to install.
- The cost to build Example 1 is $1,350,000. The cost to build Example 2 is $1,800,000.
- Thus, the marginal extra cost to build Example 2 is $450,000.
- Given the extreme storm, the repair cost for Example 1 is $938,554, and the repair cost for Example 2 is $289,155.

- The annual repair cost for Example 1 is \$9,386. The annual repair cost for Example 2 is \$2,892.
- The annual benefit (benefit = reduction in future repair costs) is \$9,386 - \$2,892 = \$6,494.

Let us say that the annual chance of the extreme storm that has been assumed to impact this particular circuit is 0.01 (once in 100 years). Assume the economic life of each tower is 100 years. Assume a discount rate of either 4%, 5%, 6% or 7%. The 7% value is the same value used by FEMA for benefit cost analyses for disaster mitigation, and is set high enough to provide good confidence that a project with a BCR greater than 1 is really worthwhile. A 4% discount rate might be applicable to some publically-owned lifeline operators, which might reflect the operator's true cost of capital, assuming it has access to tax-free bond market. The 100 year economic lifetime for transmission towers reflects that the common 50-year lifetime for buildings is almost certainly too short for power transmission towers; but also recognizes that while we do not yet have 100 years experience with operating high voltage towers anywhere in the world, de facto failure rates of towers have been more like once in 10,000+ years (excepting extreme storms) for many transmission system operators. Whether the actual towers have true lifetimes over 100 years has little impact to the analysis, as for any practical value used for the discount rate (say 7%), the change in NPV (\$1, r, 50 years) versus NPV (\$1, r, 100 years) is very small (3.4% difference from 14.27 to 13.80, see Table 7-2).

Years / Rate	4%	5%	6%	7%
50	21.48	18.26	15.76	13.80
75	23.68	19.48	16.46	14.20
100	24.50	19.85	16.62	14.27
1000	25.00	20.00	16.67	14.29

Table 7-2. Present Value Table

Following the above computations, the Net Present Value (NPV) of the accrued reduction in future losses for Example 2 versus Example 1 is (assuming r = 4%, n = 100 years) is 24.50 * \$6,494 = \$159,103.

Table 7-3 presents the Benefit Cost Ratio (BCR) for the various years / discount rates from Table 7-2. The BCR is computed as:

$$BCR = \frac{NPV(Future\ \ Benefits)}{Capital\ \ Cost_0} = \frac{\sum_{n=1}^{Nyears} Annual\ \ Benefit \Big/ (1+r)^n}{Capital\ \ Cost_0} = \frac{(Annual\ \ Benefit)(PV(r,n))}{C_0}$$

Where PV(r,n) is the present value of \$1 at discount rate r over n years (Table 7-2), and the capital cost is the marginal cost to construct a more robust circuit (Example 2) than a lower cost circuit (Example 1).

The results in Table 7-3 show clearly that for the incremental \$450,000 in capital cost, the benefits from future losses in this type of scenario storm are not high enough to warrant the capital cost. For example, assuming 7% (100 years), the benefits of building this more robust system are about 0.21 * \$450,000 = \$92,665 (Table 7-4).

Let us say that the owner is a public entity with access to low cost capital. So, say we use a 4% discount rate. The BCR is 0.35, with $159,135 of benefits. Neglecting all other storms, and neglecting all other benefits, under what conditions would Example 2 be economically worthwhile?

- Answer 1. If the incremental capital cost to build the more robust suspension towers is less than $159,135, then the BCR will be 1 (or higher).
- Answer 2. If the annual chance of this type of storm is not 0.01 (one in one hundred years), but rather 0.05 (once in 20 years), then the net present value of future benefits will be five times higher, and the BCR (100 years, 4%) will be 1.65.

Years / Rate	4%	5%	6%	7%
50	0.31	0.26	0.23	0.20
75	0.34	0.28	0.24	0.20
100	0.35	0.29	0.24	0.21
1000	0.36	0.29	0.24	0.21

Table 7-3. Benefit Cost Ratios, Single Scenario Storm, Repair Cost Only

Years / Rate	4%	5%	6%	7%
50	$139,505	$118,554	$102,358	$89,622
75	$153,781	$126,535	$106,864	$92,191
100	$159,135	$128,892	$107,914	$92,665
1000	$162,350	$129,880	$108,233	$92,771

Table 7-4. Net Present Value of Future Benefits for Single Scenario Storm, Repair Cost Only

In practice, a more robust BCR model should include the complete gamut of future storms that might affect the transmission circuit. A complete analysis would include every possible return period storm, but for practical purposes, a range of nine return periods should be sufficient to reasonably capture the effects of all storm sizes.

In order to do this calculation for various size storms, we need to create an extreme event hazard curve. Table 7-5 presents an assumed wind-speed hazard curve.

Return Period (Years)	Extreme wind speed (mph)	Annual Chance of Exceedance	Interval Probability	Tower Collapse Probabilities (DE, A, S)
10	70	0.1	0.05	0.0001, 0.001, 0.001
20	80	0.05	0.03	0.001, 0.01, 0.01
50	90	0.02	0.01	0.003, 0.05, 0.05
100	100	0.01	0.005	0.01, 0.2, 0.2
200	110	0.005	0.003	0.02, 0.4, 0.4
500	120	0.002	0.001	0.04, 0.6, 0.6
1000	130	0.001	0.0006	0.10, 0.8, 0.8
2500	140	0.0004	0.00039	0.20, 0.9, 0.9
10000	150	0.00001	0.00001	0.30, 0.99, 0.99

Table 7-5. Hazard Return Intervals

In order to do the complete benefit cost analysis over all storms, we need to compute the probability of tower damage (direct collapse due to external loads) for each of the 9 wind speeds, for each kind of tower. For purposes of development of this BCR model, let us say that this has been computed, with Examples 1 and 2 representing the tower failure probabilities for the 100 year wind (100 mph). This might be representative for towers designed for a 100 mph wind speed, following ASCE 74, with the highest stressed members at nearly Fy in tension, the buckling capacity in compression (with buckling FS = 1), or pullout of the footing from the soil (with FS = 1).

We then re-compute the tower failure probabilities and circuit reliabilities using the tower failure probabilities from the right-most column in Table 7-5. We do this for both Example 1 (9 times) and Example 2 (9 times), or 18 times total. The results are listed in Table 7-6.

Return Period (Years)	Ex. 1 Repair Cost	Ex. 2 Repair Cost	Ex.1 N (Col J)	Ex. 2 N (Col J)	Ex.1 Reliability	Ex. 2 Reliability
10	5,303	1,466	0.051	0.009	0.951	0.991
20	53,030	14,659	0.506	0.094	0.494	0.911
50	262,510	72,490	2.507	0.464	0.082	0.629
100	938,554	289,155	8.916	1.853	0.000	0.157
200	958,400	578,311	9.042	3.706	0.000	0.025
500	966,400	871,497	9.082	5.579	0.000	0.004
1000	990,400	1,180,804	9.202	7.533	0.000	0.001
2500	1,030,400	1,363,669	9.402	8.651	0.000	0.000
10000	1,070,400	1,520,400	9.602	9.602	0.000	0.000

Table 7-6. Scenario Losses, Examples 1 and 2, Different Return Periods

Table 7-6 shows that for the more common extreme storms (return periods of 10, 20 and 50 years), the Example 2 tower damage is about 80% lower, and reliability is higher.

Similarly, Example 2 has much lower expected repair costs, up to about the 500 year return period storm; but for rarer storms with even higher wind speeds (130 mph or higher), since the lateral loads control (130 mph being much higher than the target 100 mph design wind speed), the repair costs reflect damage to the more expensive towers.

Table 7-6 also shows us that by increasing the capacity of intermediate suspension towers to resist unbalanced loads, we materially increase the circuit performance for wind speeds from about 70% to 100% of the design wind speed and somewhat higher. But for wind speeds of 40% or more than the design wind speed, increasing the unbalanced load carrying capacity of the suspension / angle towers makes little overall difference.

Using the data from Tables 7-5 and 7-6, we can compute the annual tower repair costs for each of the 9 storm intervals, for both Examples 1 and 2. The results are in Table 7-7.

Return Period (Years)	Ex. 1 Repair Cost	Ex. 2 Repair Cost	Ex 1 – Ex 2 Repair Cost
10	$265	$73	$192
20	$1,591	$440	$1,151
50	$2,625	$725	$1,900
100	$4,693	$1,446	$3,247
200	$2,875	$1,735	$1,140
500	$966	$871	$95
1000	$594	$708	$(114)
2500	$309	$409	$(100)
10000	$107	$152	$(45)
TOTAL	$14,026	$6,560	$7,466

Table 7-7. Annualized Losses, Examples 1 and 2

Table 7-7 shows that most of the reduction in annualized tower repair cost losses are for storms in the 10 to 500 year return period ranges. For truly extreme storms (1,000 to 10,000 year events), the strategy of reinforcing the suspension towers for cable break loads, while still keeping design target wind speeds of 100 mph, yields *negative* benefits. Still, when summing up over all extreme storms, the net annualized reduction (savings) of tower damage is positive, $7,466.

Tables 7-8 and 7-9 provide the Net Present Value of the Example 2 upgrade strategy. For this example, we see that when assuming (100 years, 4%), the BCR is 0.41. In other words, if the extra capital cost for Example 2 is $450,000, and we include only benefits for reduction in tower damage, the BCR of 0.41 shows that this project is not worthwhile on its own. Another way of looking at this is that if the capital cost were reduced by 60% to $180,000, then the BCR (100 years, 4%) would be 1.02, and the upgrade of the suspension towers would be marginally worthwhile.

Years / Rate	4%	5%	6%	7%
50	0.36	0.30	0.26	0.23
75	0.39	0.32	0.27	0.24
100	0.41	0.33	0.28	0.24
1000	0.41	0.33	0.28	0.24

Table 7-8. Benefit Cost Ratios, All Storms, Repair Cost Only

Years / Rate	4%	5%	6%	7%
50	$160,386	$136,299	$117,678	$103,036
75	$176,798	$145,475	$122,859	$105,990
100	$182,954	$148,184	$124,067	$106,534
1000	$186,650	$149,320	$124,433	$106,657

Table 7-9. Net Present Value of Future Benefits for All Storms, Repair Cost Only

Other Economic Benefits of Mitigation

The BCA example in Section 7.2 examines only the marginal extra cost to build "stronger" towers, and the marginal benefits from reduced future repair costs for these "stronger" towers. This approach is reasonable for collapses to a single circuit in a N-1 network in a single storm. However, this approach ignores other economic impacts which are important whenever damage results in less than N reliability.

The underlying economic impacts due to a storm vary widely. High winds can damage towers, as well as almost everything else in the community. For example, high winds can damage single family houses; break windows in high rise office towers; blow down timber; and ruin crops. Ice accumulation can damage trees. Coincident damage caused by wind and ice includes flooding, either due to rising waters due to inadequate drainage from falling rain / ice-jammed rivers; storm surge; failed levees; intense rainfall-induced landslides; etc.

Some (but not all) high voltage transmission operators are also retailers (deliver power to end users). The storm that causes damage to the high voltage transmission system (generally 115 kV and higher) is likely to also cause damage to the sub-transmission (generally 60 kV – 96 kV) and low voltage (generally 34.5 kV and lower) distribution system. It is not unreasonable to say "what is the point of upgrading the high voltage system if there is concurrent damage to the lower voltage system?" To properly answer this important question would require a model that forecasts the damage to the distribution system, as well as the capacity of the utility to make repairs (size of repair crews, mutual aid, spare parts and equipment).

In order to simplify this complex situation into a manageable problem, one could use the following simplifying approach:

- All other utilities / property owners have taken (or will have taken) suitable mitigation actions to reduce the damage to their systems / properties to the point that the high voltage transmission system is the critical system in terms of restoring power.

- For example. Assume a storm damages all high voltage circuits that serve a community of 100,000 people. Assume there is no local generation for this community. Assume that the same storm that damages the high voltage circuits also causes some damage in the community, including damage to the low voltage distribution system. Assume that the damage to the high voltage circuits requires 3 days of work to restore average day load capacity to the community (this requires knowledge of the total amount of damage, total repair crew capability and assumptions about restoration priorities). Assume that the damage to the low voltage distribution system takes 4 hours to restore power to 100% of customers.

- Then, we make the following observation. The impact of the high voltage circuit failure results in 2.83 days of 100% power outage (= 3 days less 4 hours). If restoration of the distribution circuit is more gradual, then a refinement can be made to reflect such gradual restoration.

- Assume that loss of power results in 90% reduction in economic activity to the community. This is based on prior economic studies of the importance of various inputs to total outputs, but restricted *to the loss of only the power input.* In other words, if there is a coincident loss of water supply for 3 days as well as power supply for 3 days, one cannot lose more than 100% of the total economic activity, yet loss of either (on their own) will lead to 75% to 90% loss. So, we implicitly assume that the water department has taken suitable mitigation steps to survive the same storm, with no more than a 4 hour outage to 100% of its customers. Clearly, this overestimates the gross benefits of mitigation of transmission towers should the water utility sustain damage that takes 3 days to repair; but at least it is rational and provides a first order estimated of the marginal benefits afforded by transmission tower mitigation activities.

- The computation of Customer Days Lost (CDLs) should be done by a person knowledgeable about the redundancy of the network, the forecast damage; and the agency's ability to make repairs.

- Assume that the economic activity per person is the daily Gross Regional Product (GRP) divided by the population served. For example, if the Gross Domestic Product of the USA is $15 trillion per year, then for the 300,000,000 people in the USA the average is $50,000 per year, or about $137 per day (= $50,000 / 365 days). Depending on the local economic conditions, this rate of $137 per day might be 100% higher (such as in Silicon Valley) or 67% lower (such as in rural farming areas), and the user should always factor in the analysis the local economic conditions. Local economic conditions can be estimated using data for each sector of the economy using data from the US Census or the Canadian Economic Accounts Quarterly Review (for the province of Quebec, the value is $136 Cdn using 2008 dollars).

- Then, the GRP loss due to a 3 day outage is simply estimated as 0.9 * $137 * 100,000 people * 2.83 days, = $34,900,000. If using FEMA default data, one would use $87 per capita per day (2003 data), inflated to year 2011, and the multiplying as follows: $87 * inflation * 100,000 * 2.83, which is about the same as the above.

- Assume that the mitigation strategy is to harden all the weakest towers along the transmission line from 100 mph to 125 mph. Assume that by doing this upgrade, that the reliability scenario analysis shows the likely damage in the same storm is much reduced (but not eliminated), such that the expected outage time is under 4 hours. In other words, if it takes 24 hours to repair one tower, then this scenario storm results in under 1/6 chance that a single tower will fail (excluding cascades). Note: if the mitigation is even more effective, then there is no incremental economic benefit as the residual damage to the other infrastructure in the community controls.

- Assume this storm has a 100 year return period.

- Then, the annualized economic benefit is $34,900,000 * 0.01 = $349,000 per year. Assume a 7% discount rate and 50 year life. The NPV is $349,000 * 13.80 = $4,816,200.

- Assume the capital cost to upgrade the towers is $4,000,000.
- Then, the BCR due to the economic impact is $4,816,200 / $4,000,000 = 1.20
- An alternative approach to computation of the economic impact of loss of power is to take the direct losses due to damage to the towers (actual repair costs) and multiply by 5. FEMA accepts this as an acceptable proxy, and calls it a "continuity premium" for essential utility service. Thus, the total losses would be 6 times the direct repair costs. Clearly, this is a gross simplification, but is very simple to implement.

The benefit cost model described above is often used by agencies such as FEMA to examine mitigation projects. The overall approach is as follows:

- Step 1. Perform a scenario storm analysis for an entire transmission circuit. Repeat this nine times, representing 10, 20, 50, 100, 200, 500, 1,000, 2,500, and 10,000 year return period storms. Compute the losses (tower damage, revenue loss, economic impacts, life safety impacts, etc.) for each of the nine storms.
- Step 2. Multiply the losses for each storm by the interval probability of the storm. By interval probability, we mean the annual chance of a 10 to 19 year storm, a 20 to 49 year storm, etc. This must be done to avoid double counting the longer return period storms, as that would show up in all lesser-return period storms. Add up the annual losses over the five storms.
- Step 3. Repeat steps 1 + 2 for the transmission line in its mitigated condition.
- Step 4. Subtract the mitigated losses (Step 3) from the unmitigated losses (Step 2). These are the annual benefits.
- Step 5. Multiply the annual benefits by the NPV of $1 at discount rate r for n years. For the common assumption of 7% discount rate and 50 year lifetime, this is 13.80 (Table 7-2). This is the NPV of all benefits.
- Step 6. Divide the NPV of benefits (Step 5) by the capital cost. This is the BCR. If the BCR is greater than 1, the project is worthwhile. If the BCR is less than 1, the project is not worthwhile unless there are some other non-quantified benefits that have not been accounted for in the analysis.

A simple refinement for the economic impacts due to loss of a transmission circuit, to factor in the actual capacity of the circuit, could be as follows:

- Obtain the maximum (non-emergency) power for the circuit in question, in terms of MVA. Most utilities have this type of information. If not, assume MVA = 700 amps * kV (up to 154 kV); 1,000 amps * kV (up to 287 kV); 1,500 amps times kV (over 300 kV).

- Assume that 1000 MVA supplies 600,000 people.
- Perform the rest of the analysis as above.

8.0 EXTREME EVENT REVIEW

The following sections provide empirical information for failures to high voltage transmission system towers under wind and ice events.

8.1 Transpower Transmission Tower Historical Reliability

The long term empirical evidence for wind-induced tower failures in New Zealand has been documented by Transpower (2006). The definition of tower failure used in this study is: when a structural member or the foundation of a tower is damaged causing the tower to either totally or partially collapse. The study presented data for towers failed due to high winds (no ice failure data is available). The key statistics are as follows:

- Transpower has 2,530 towers in service in 1946, increasing to 24,177 towers by 2006. The total number of "tower-years," based on actual number of towers to have been built in selected years, is about 1,046,000 tower-years (all steel towers).

- The total number of observed tower failures due to high winds has been 54, of which 41 were structural failures, and 13 were foundation failures.

- The historical record is not clear enough to precisely discern between initial tower failures and "pull down" tower failures. Based on the available data, we estimate that between 58% and 70% of the failures were "initial failures," and the remainder "pull down" failures of adjacent towers. Twenty-three of the 54 failures were on adjacent towers, meaning that about 58% were "initial failures," and as many as 42% were "pull down" failures. However, given that the high winds were possibly applied to adjacent towers at nearly the same time, we think it more prudent to say that pull down failures were likely between 30% and 42% of all the failures.

- The known causes of tower structural damage were due to overloads from winds (not impacts from tree falls). Three of the thirteen foundation failures were due to water erosion over time that resulted in weakened foundations as compared to originals.

- The Transpower wind design philosophy has varied over time. For conductors, the equivalent design wind speed has been as low as 72 mph (towers built from 1957 to 1963) to 90 mph (common for other construction years). About 40 of the 54 towers failures were for those built to the 72 mph criteria. Given the age of construction, there have been about 190,000 tower-years in service for those towers with conductor loads designed to the 72 mph criteria.

- Therefore, we get the following Transpower tower reliability data.

    - Failure Rate/Year (all towers). 0.0000516 tower failures / year

    - Failure Rate/Year (72 mph towers only). 0.000211 tower failures / year

  - Failure Rate/Year (90 mph towers only). 0.000016 tower failures / year

- Transpower noted the higher failure rate of the towers designed for 72 mph conductor loads. In 1986, and again in 1990-91, Transpower upgraded many of these 72-mph-designed towers so that they could withstand higher wind speeds; however, in spite of these two stages of upgrading, 3 towers failed in January 2004 due to high wind.

- The rate of failure decreased by 13 times by changing the design criteria from 72 mph to 90 mph. If load is proportional to the square of wind velocity, then we observe that for a (90/72)^2 increase in nominal design strength (56% strength increase), the probability of failure reduces by 13 times. This indicates that the tower fragility model, if using wind velocity as the independent variable, should show a 13-fold increase in failure rate for a 25% increase in velocity; or a 13-fold increase in failure rate for a 56% increase in total wind force.

The actual wind speeds on the tower and conductors at the times of the Transpower collapses are not known (no anemometer data). However, study of the some of the tower failures suggested the following: 3 failures at estimated wind speed in excess of 143 mph (localized topographic effects resulting in high wind speeds); 10 failures at estimated wind speed in excess of 100 mph; 14 towers failed at estimated wind speeds of gale intensity (47 to 63 mph); 8 towers failed at estimated wind speeds of severe storm intensity (55 to 72 mph).

Note that for Transpower, 22 towers failed at estimated wind speeds below even the lowest design wind speed (72 mph). This can be explained, if any of the following are true:

- The actual wind speeds were higher than those estimated
- The actual towers were not installed as per the design. For example, some angles might have been bend, bolts not installed correctly, etc. leading to innate weaknesses
- The original designs under predicted actual loads on the tower elements. For example, simplified "truss" type models ignore eccentricities on compression elements and their joints, and could under predict true stresses.
- Accumulated corrosion / age effects weakened the towers
- The actual spans and line angles in the field were longer / sharper angles than allowed for in the design
- The actual conductors in the field were large (heavier) than those assumed in the design

One item we learn from review of the Transpower data is that it is almost certain that some towers can collapse as wind speeds up to 17 mph below the design wind speed, or perhaps just 58% (= 55 / 72)^2 of the design wind force. As discussed in Section 4, we handle this by using a truncated lognormal distribution, whereby there is zero chance of tower failure below a prescribed wind speed (or wind force level).

8.2 Storm Database

An electronic database is not provided in this report. Contact G&E Engineering Systems Inc. if you wish to obtain a copy of the database.

This database is populated with 55,439 tornadoes, 299,817 wind storms, and 266,869 hail storms. Most data is from 1950 through 2010, and applies to the United States. The source data is from the Severe Weather Database (NOAA).

The database includes the following information (most field are populated; data not verified).

- Maps showing the assumed linear path (true path would be somewhat different) with background terrain, satellite imagery or roadway networks. A placeholder for the user to enter a photograph (no photos were inserted into the database). A zoom control feature. All maps require on-line internet access, and are subject to restrictions by Google; which current allow up to (about) 1,000 maps per IP address per day; if you request too many maps, the map field will report "unavailable" or similar; so try again later! The "S" symbol in the maps refers to the starting latitude / longitude. The "E" symbol in the maps refers to the ending latitude / longitude.
- Date and time and time zone for the storm.
- State (2-letter abbreviation) and State FIPS code.
- Injuries.
- Fatalities.
- Property losses (historical dollars)
- For storms covering multiple states, field to list the states and counties affected.
- A comment field that the user can enter text discussion about the effects of the storm. The text field is virtually unlimited (up to 2 GB).

For each type of storm, the following additional information is provided:

- Tornadoes: Starting latitude, longitude; ending latitude / longitude. Fujita scale (EF scale after Jan 2007). Length on the ground, in miles. Width in yards (generally widest areas, as observed).
- Wind Storms: maximum reported wind speed, in knots. Note: this database does not include historical hurricanes.
- Hail storms. maximum reported hail size, in inches.

8.3 Ice Storm Database

An electronic database is not provided in this report. Contact G&E Engineering Systems Inc. if you wish to obtain a copy of the database.

This database is populated with 28 ice storms that have occurred in the USA and Canada between 1948 and 1998, primarily concentrating on the USA Pacific Northwest, and select storms along the St. Lawrence Valley.

The user can add ice storm data into the database as it occurs. The intent of this database is to provide each electric utility with a simple-to-use and consistent way to accumulate empirical evidence of ice-storm damage.

For each ice storm event, the following is provided:

- Storm_ID. This is a unique number used by the database. To create a new record, select "Records->New Record" or the keyboard equivalent.
- Start Date. This is the first day of the storm that ice started to accrete (or the wind started to blow hard) onto natural and man-made structures. For many storms, this is the same as the End Date.
- End Date. This is the last day of the storm where ice continued to accrete or the wind continued to blow hard. For some storms, this might be 2, 3, 4, 5 or even 6 days after the Start Date.
- Description. This is a text description of the effects of the ice storm. The information provided is a combination of hazard (amounts of ice, wind velocity) and impacts from wind and ice to electric utility infrastructure. For large storm events (like the storm event from Jan 4 1998 to Jan 20, 1998), the description is lengthy. For many other storms, the description is short. Reference material is listed.
- Map. This is a map showing the location of the storm. Most maps in the database were extracted from a database developed by Kathleen Jones.
- Photos 1 and 2. The database includes places to add in up to two photos for the storm. Photo can be in .jpg, .gif, .pdf, .bmp, .png and several other formats. To insert a photo, right click within the photo place-holder, and then navigate to the location on a computer with the photo. To maintain database portability, it is recommended that the photo be inserted into the database; to limit the size of the database, only a reference to the photo can be chosen (but if the database or photo is moved to another folder, the database will not be able to show the photo).

8.4 Seismic Database

An electronic database is not provided in this report. Contact G&E Engineering Systems Inc. if you wish to obtain a copy of the database.

This database is populated with seismic damage information at substations 57 substations for earthquakes that have occurred, worldwide, from 1952 (Taft, California) to 2008 (Sichuan, China).

The database has three layouts:

- Utility
- Substation
- Damage

This database is populated with 606 records of damage that occurred at 57 high voltage substations.

WARNING: All databases are provided in an "as-is" condition. Data and photos in the databases are not your property. All photos and maps in the databases should be assumed to have been copyrighted by third parties. Nothing in this report conveys any ownership of any sort of this data to you. You are responsible to maintain and propagate the creative commons license. If you do not agree to respect the creative commons license, you may not use the databases.

Layout: Utility.

Name: enter the name of the utility.

Map: insert a map of the service area for the utility.

It is recognized that maps for many high voltage power transmission utilities are no longer "public domain". It is suggested that a simplified map be provided, to allow the user to know roughly where the utility is located. It is not necessary to fill this information.

Layout: Substation.

Name: enter the name of the substation. A drop down list is provided that includes all the names of substations already in the database. It is preferred that the name be used from the drop-down list, so that better searches can be made (for example: find all records at substation "name").

Note: if this substation was damaged by more than one earthquake, enter a separate record for the substation (use a slightly different substation name, so that the database can associate different earthquake information with the same substation), and enter a separate earthquake (next field).

Earthquake: enter the common name for the earthquake and magnitude. A drop down list is provided that includes all the names of earthquakes already in the database. It is preferred that the name be used from the drop-down list, so that better searches can be made (for example: find all records from earthquake "name"). When entering a new name, it is suggested to use the following format:

- Name (brief, under 15 letters)
- Magnitude (for example: M 6.8)
- Year (for example: 1971)

PGA: enter the estimated peak ground acceleration (PGA) level or recorded PGA at the substation. If information is available for horizontal and vertical motions, enter both.

Comments: Describe the overall damage to the substation, basis of the PGA levels, references for further information. Since damage information for each individual piece of equipment can be entered separately, include substation-wide descriptions.

Owner: List the name of the owner (for example: LADWP).

Latitude: Enter the latitude of the substation, in decimal degrees (4 decimal points is usually sufficient). For locations north of the equator, enter a positive value. Example: 34.31

Longitude: Enter the longitude of the substation, in decimal degrees (4 decimal points is usually sufficient). For locations in Canada and the USA, enter a negative value for west longitude. Example: -118.4882

Tabbed Maps and Drawings

There are 8 tabs that allow the user to examine various types of maps and enter substation-specific information.

Zoom. The Zoom control allows the user to select between the available zoom levels. The "Terrain", "Hybrid" and "Feature" maps are only available if the user is connected to the internet, and if Google has the data and allows the map to be shown. Usually, set the Zoom to 15, which is the closest zoom-in to observe terrain information. Google will usually allow about 1,000 maps to be served daily. Google's policy may change at any time.

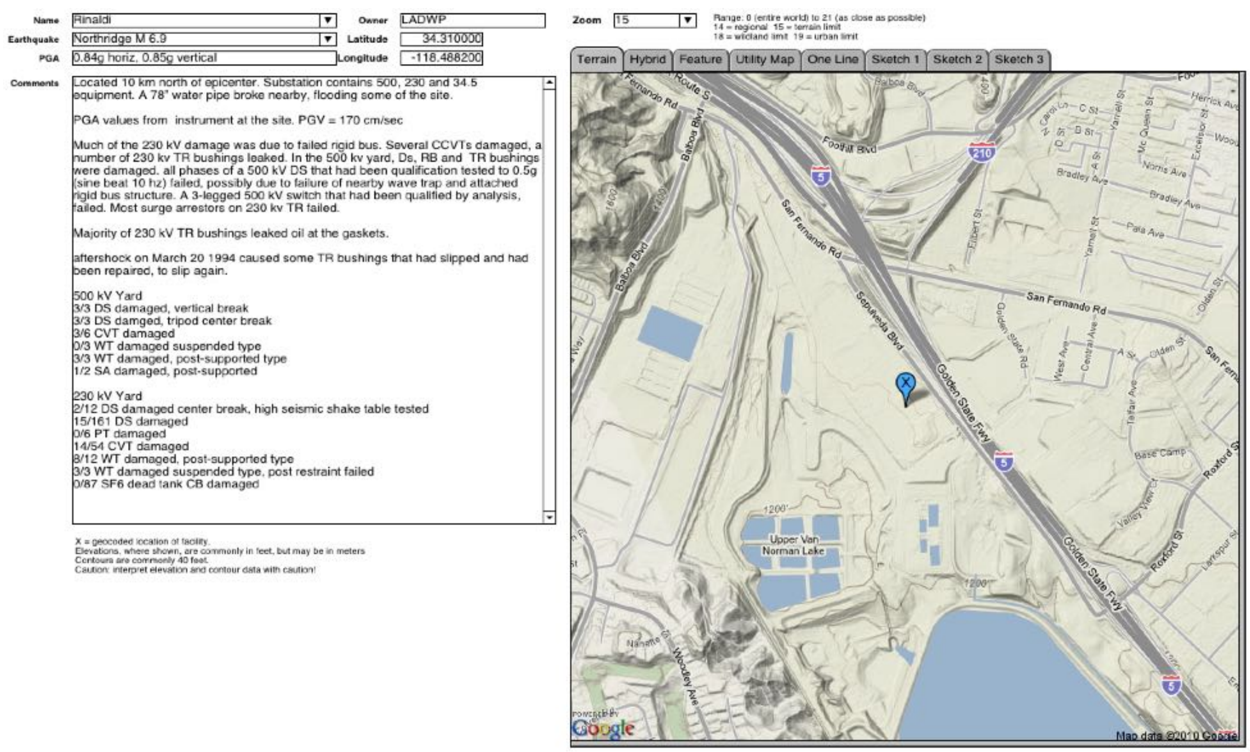

Figure 8-1 Seismic Database: Substation Layout

Terrain. This map shows a blue "X" at the latitude / longitude given. The terrain is shown in terms of 40-foot contours (may be in meters in some places). Zoom = 15 will usually provide the highest resolution map. If no latitude / longitude information is supplied, the database will often return blank maps, or perhaps something at the equator at the 0 meridian.

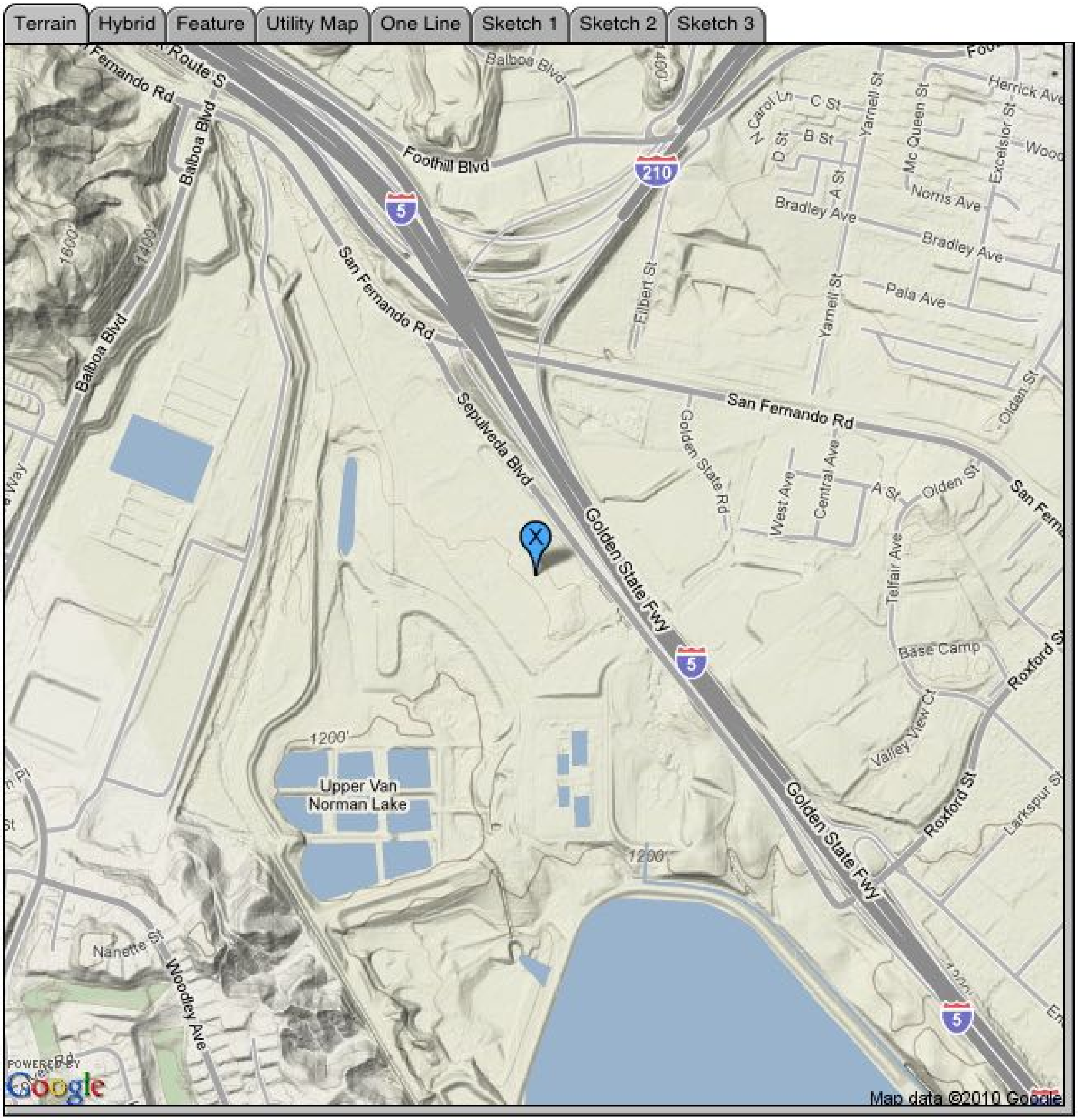

Figure 8-2 Seismic Database: Substation Layout (Terrain Map)

Hybrid. This map shows a blue "X" at the latitude / longitude given. The hybrid map shows an aerial photo of the area, overlaid by major cultural features, such as major roads and highways; often named. Zoom = Closest will usually provide a close-in view in urban areas; in rural areas, the closest view may be Zoom = 18 (or so).

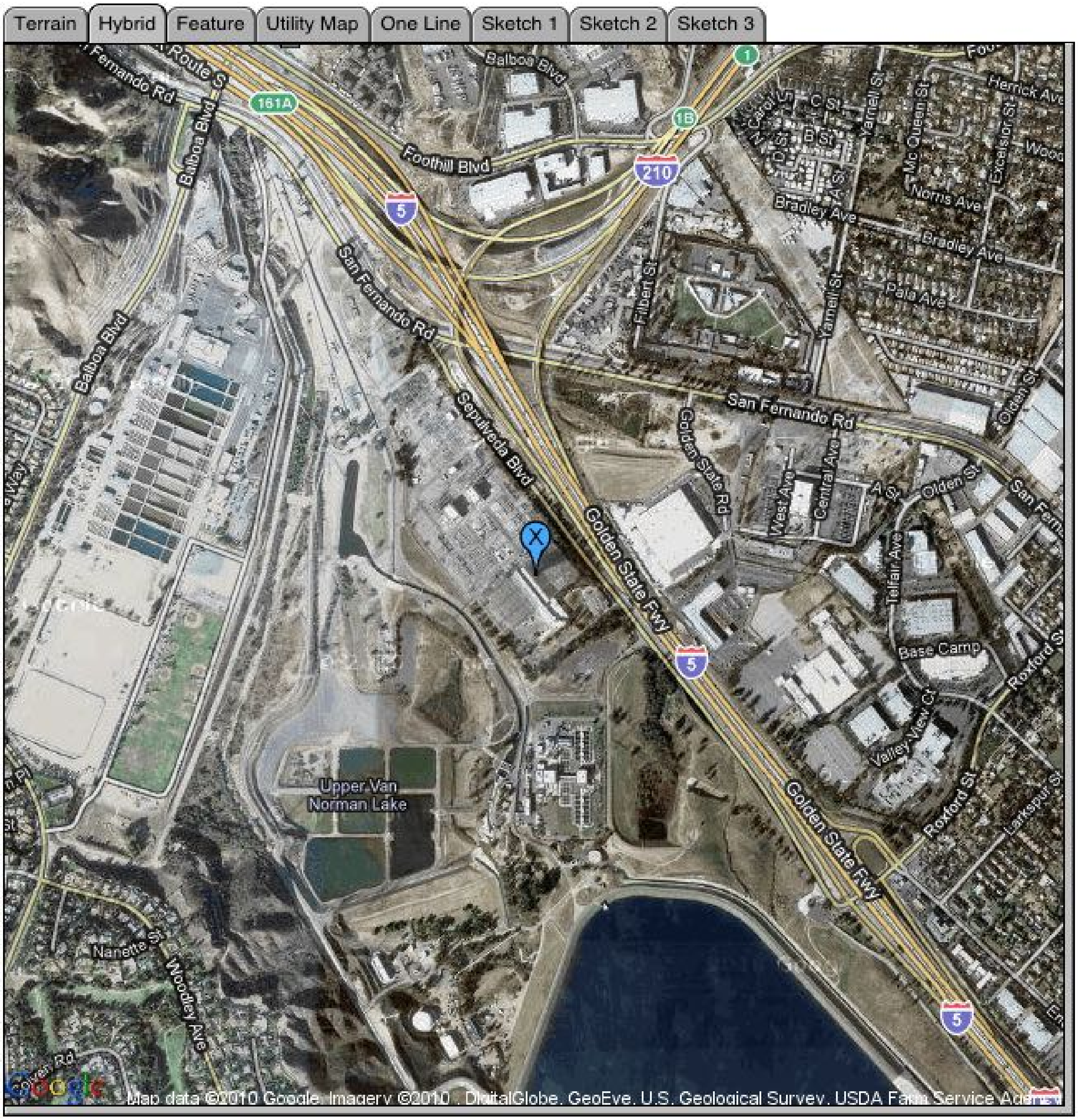

Figure 8-3 Seismic Database: Substation Layout (Hybrid Map)

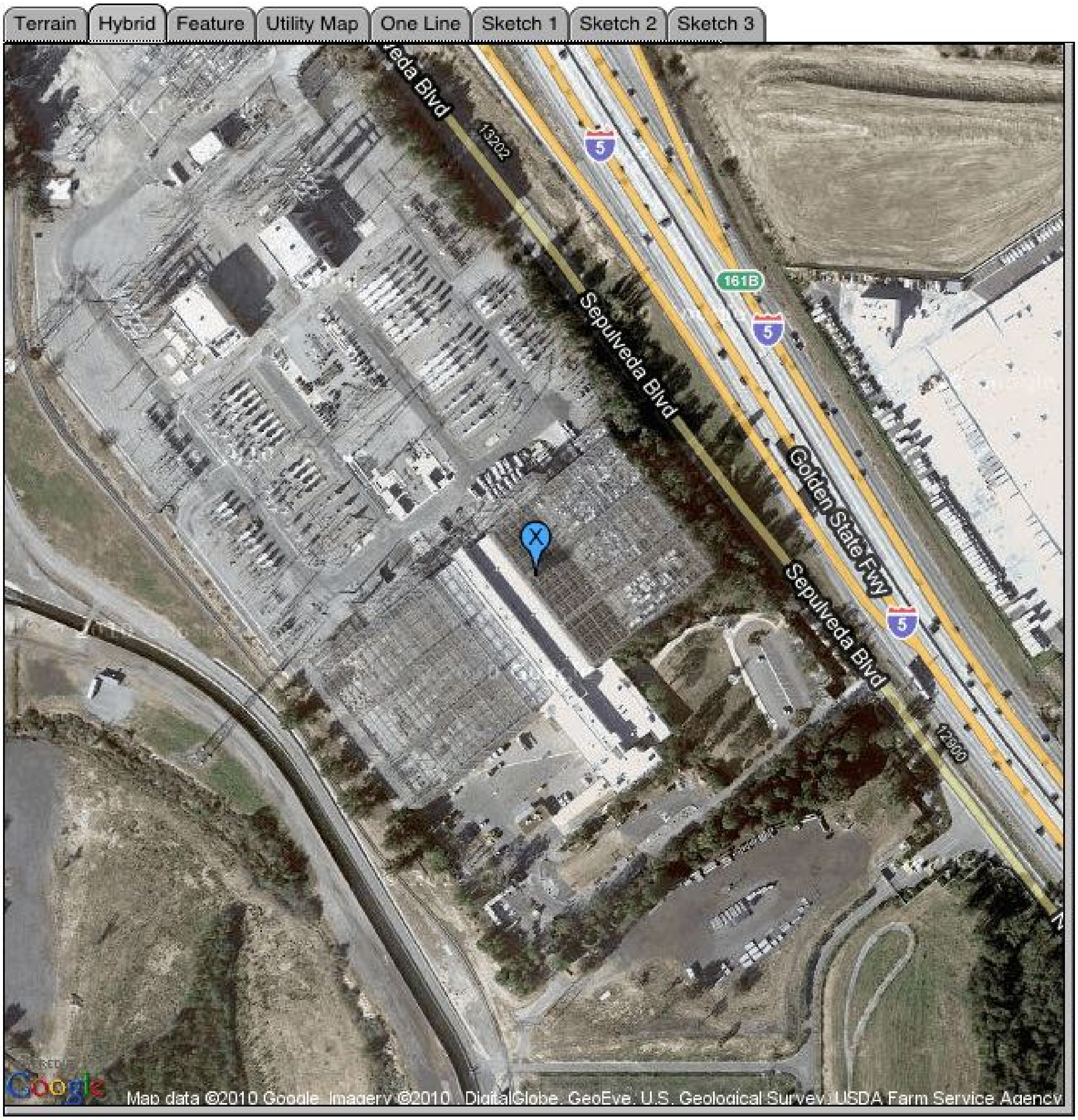

Figure 8-4 Seismic Database: Substation Layout (Close-In Hybrid Map)

Feature. The feature map shows a general view of the area, showing only features such as roads, highways, parks, bodies of water, etc. Zoom = 14 will usually provide a suitable view in urban areas.

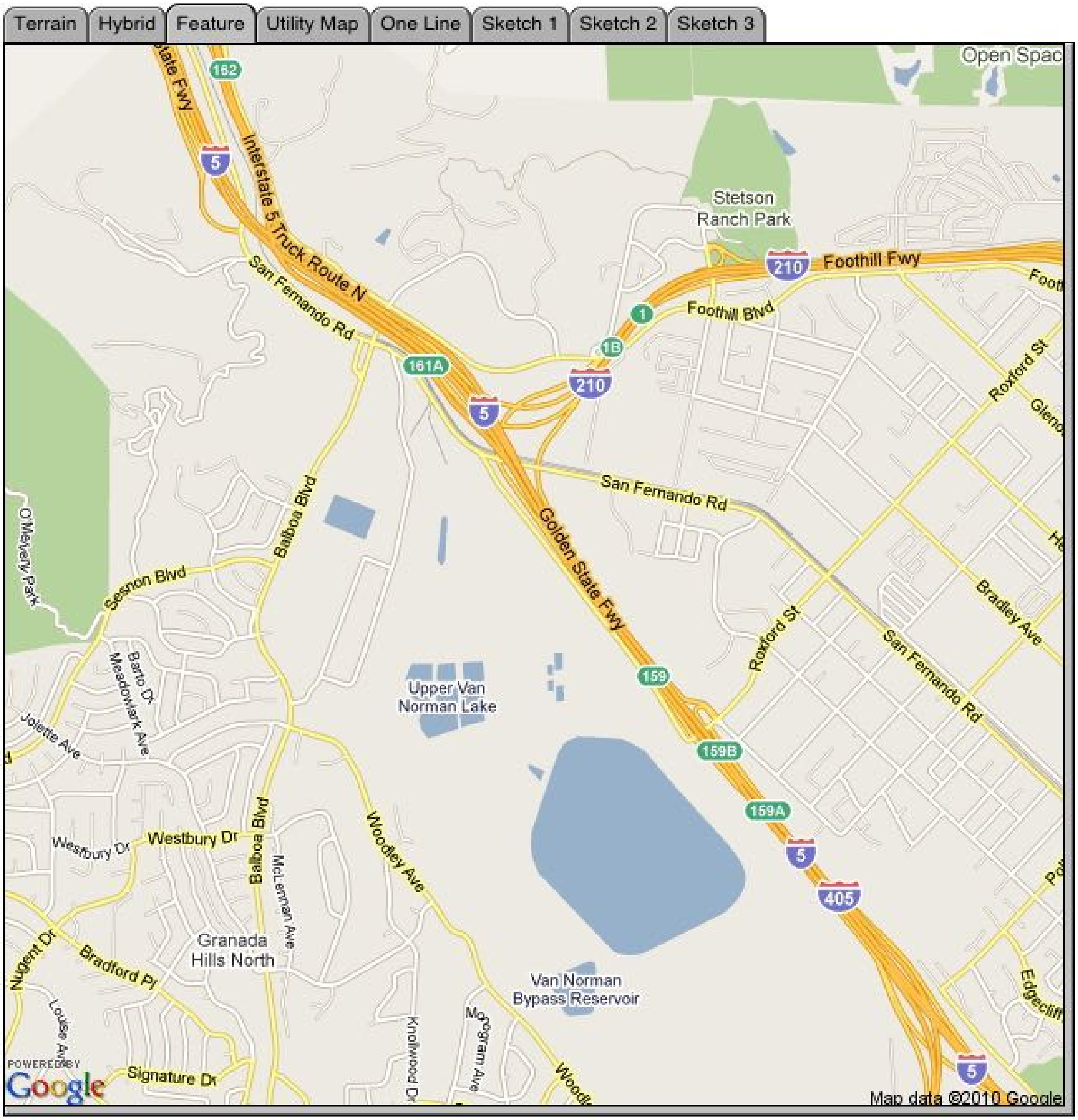

Figure 8-5 Seismic Database: Substation Layout (Feature Map)

Utility Map. If the user entered a map for the specific utility listed at the "Owner", the map will be shown. In order to enter this map into the database, it must be accessed from the "Utility" layout.

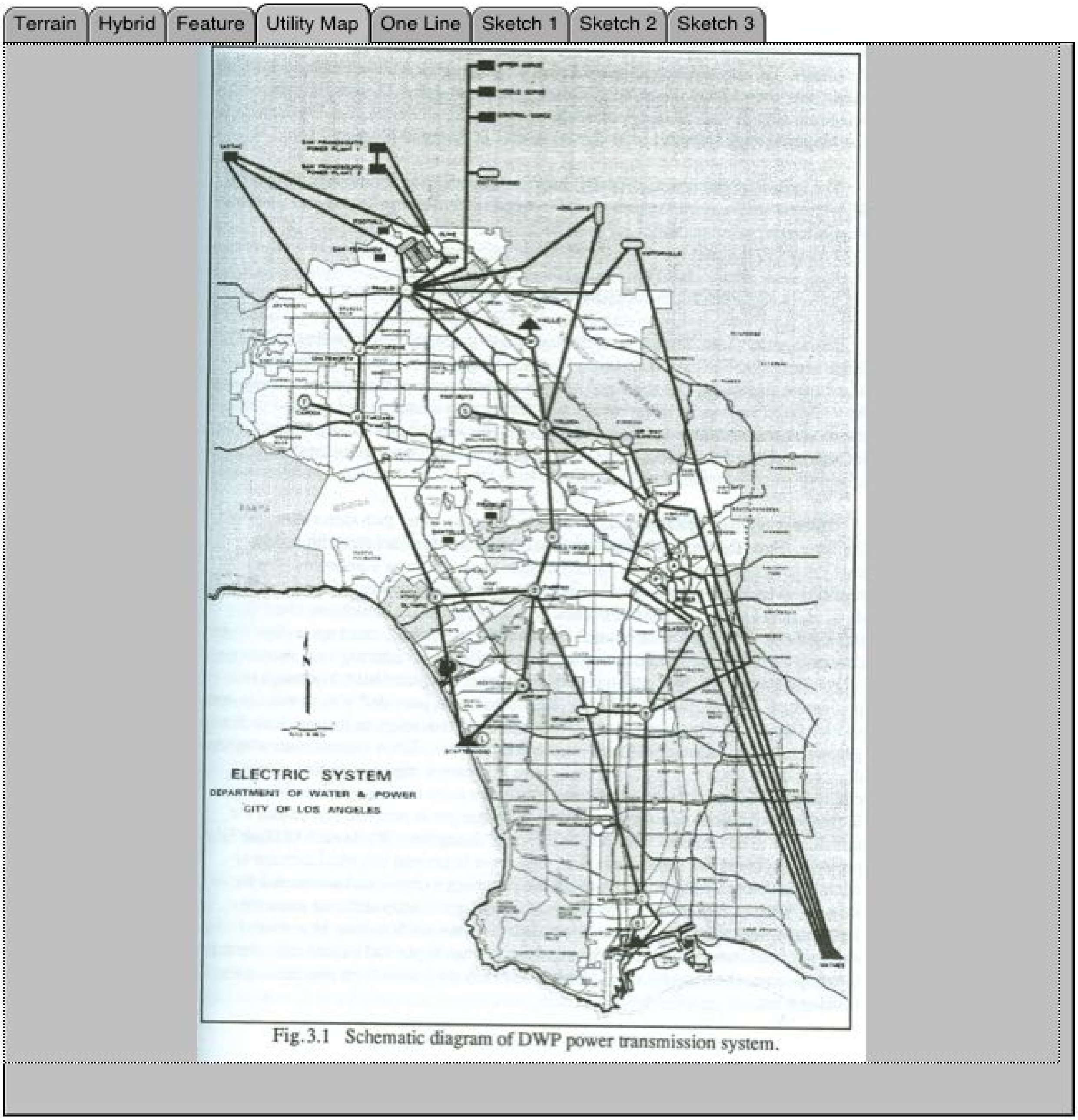

Figure 8-6 Seismic Database: Substation Layout (Utility Map)

One Line, Sketch 1, Sketch 2, Sketch 3. The user can enter any graphical information about this substation into these four tabs. The tab "One Line" is the suggested location to put in a one-line diagram, if available. The data can include images in many formats, including .jpg, ,png, .gif, .bmp, etc.

Layout: Damage.

Name: enter the name of the substation.

Map: insert a map of the service area for the utility.

It is recognized that maps for many high voltage power transmission utilities are no longer "public domain". It is suggested that a simplified map be provided, to allow the user to know roughly where the utility is located. It is not necessary to fill this information.

Layout: Substation.

Name: enter the name of the substation. A drop down list is provided that includes all the names of substations already in the database. It is preferred that the name be used from the drop-down list, so that better searches can be made (for example: find all records at substation "name").

Earthquake: (blue box). The database will automatically fill in the name of the earthquake that is associated with this substation. Since there may be multiple earthquakes that affected one substation, you must use a different substation name (in the Substation layout) for each earthquake.

PGA: (blue box). The database will automatically fill in the PGA information that is associated with this substation for this earthquake.

Photographer: Enter the name of the person (or organization) who took the photo and who may have the copyright for the photo. If you wish to publish the photo, you must first obtain the copyright release from that person. If you do not obtain the copyright release, and you publish or otherwise use or re-use this information, you agree to, and accept, and you are personally responsible for any legal actions.

Photo Date: Enter the date the photo was taken. This may be different than the day of the earthquake.

Earthquake: (black box). Select the name of the earthquake which cause the damage observed in the photo. This may be left blank if the information in the Earthquake (blue box) is correct. If the user has not entered a unique substation name in the substation layout, then enter this information here.

Photos: Enter up to two photos for this piece of equipment. If you have more photos that you want to insert, then create a new record.

Equipment: Describe the type of equipment (long name). This may be left blank.

Eq_Abbr: Enter the two-letter abbreviations for each type of equipment in the photo of interest. For example: TR = transformer.

Voltage: Enter the voltage (high side) for this piece of equipment, in kV. For example, enter 230 for 230,000 volts. A drop-down box allows the user to select common voltage levels.

Comment: Enter a description of what happened to the piece of equipment (optional). Enter as much information as you like. Add in references for more information.

9.0 REFERENCES

ASCE, Wind Forces on Structures, 1961.

ASCE 7, Minimum Design Loads for Buildings and Other Structures, 7-98, 7-05, 7-10, 1998-2010.

ASCE 74, Guidelines for Electrical Transmission Line Structural Loading, ASCE Manual 74, 2008.

Campbell, M., Failure of 345 kV Steel Transmission Structures, New Brunswick Power Corporation, Canada, presented at International Symposium on Very High Voltage Networks, Sibiu, Romania, May 31 - June 3, 1995.

Durst, C.S., Wind Speeds Over Short Periods of Time, *Meteorological Magazine*, Vol. 89, London, England, 6 pp, 1960.

Richards, W., Ice Storm 2003 in New Brunswick, February 203, 2003, Environment Canada.

Fujita, T., Proposed Characterization of Tornadoes and Hurricanes by Area and Intensity. Satellite and Mesometeorology Research Project Report 91, University of Chicago, 1971.

Isyumov, N., The Aeroelastic Modeling of Tall Buildings, Proceedings of the International Workshop on Wind Tunnel Modeling Criteria and Techniques in Civil Engineering Applications, National Bureau of Standards, Gaithersburg, MD, 1982.

Jones, K., Thorkildson, R., Lott, N., The Development of a U.S. Climatology of Extreme Ice Loads, NOAA /NESDIS, National Climatic Data Center, TR2002-01, December, 2002.

National Bureau of Standards, Hurricane Wind Speeds in the USA, 1980.

Simiu, E., and Scanlan, R.H., Wind Effects on Structures, Wyle and Sons, 459 p., 1978.

SEAW/ATC, SEAW Commentary on Wind Code Provisions, SEAW/ATC-60, Prepared by the Structural Engineers Association of Washington, published by the Applied Technology Council, 2004.

Texas Tech University, A Recommendation for an Enhanced Fujita Scale (EF-Scale), Wind Science and Engineering Center, Rev. 1. January 26, 2006.

Transpower, Tower Failures in New Zealand, Document ADGL.400.14 Issue 2, D6, June, 2006.

U.S Department of Agriculture, Rural Utilities Service, Design Manual for High Voltage Transmission Lines, RUS Bulletin 1724E-200, U.S Government Printing Office, Washington, DC, 2004.

WIND MAPS

Appendix A describes various wind maps that can be used as input to reliability analyses.

Unless otherwise described, these maps specifically exclude increases in wind speeds at cliffs, escarpments, at the crests of isolated hills and ridges. For example, in the Canada wind map (Morris, 2009), the underlying source weather station dataset was revised to specifically exclude weather stations affected by topography, with the net effect of *reducing* the wind speeds in the map, by about 9 mph. Other sections of this report describe how these local topographic wind speed-up effects can be accounted for.

A.1 Wind Speed Map - Canada

The basic wind speed map for Canada is shown in Figure A-1.

Figure A-1 is drawn as follows:

- The map is drawn in a geographical coordinate system (GCS_North American_1983, Datum D_North American_1983, Prime Meridian: 0). For example, the international boundary between the mainland USA and Canada is at latitude 49.0 degrees, as evidenced by the termination of the wind contours along a horizontal line for British Columbia, Alberta, Saskatchewan and Manitoba.
- The plotted area ranges from western Yukon to eastern Newfoundland, and from the Lake Erie international boundary south of Windsor to Alert, in northern Ellesmere Island.
- The map includes 43 wind-speed contours (in km/hr).

A convenient projection for both Canada and USA is NAD-1983_UTM_Zone15N. Projection is Transverse Mercator, False Easting: 500000, False Northing : 0, Central Meridian: -93.0; Scale Factor 0.996, Latitude of Origin: 0.0. Figure A-2 shows the same map as in Figure A-1, but using the UTM 15N projection.

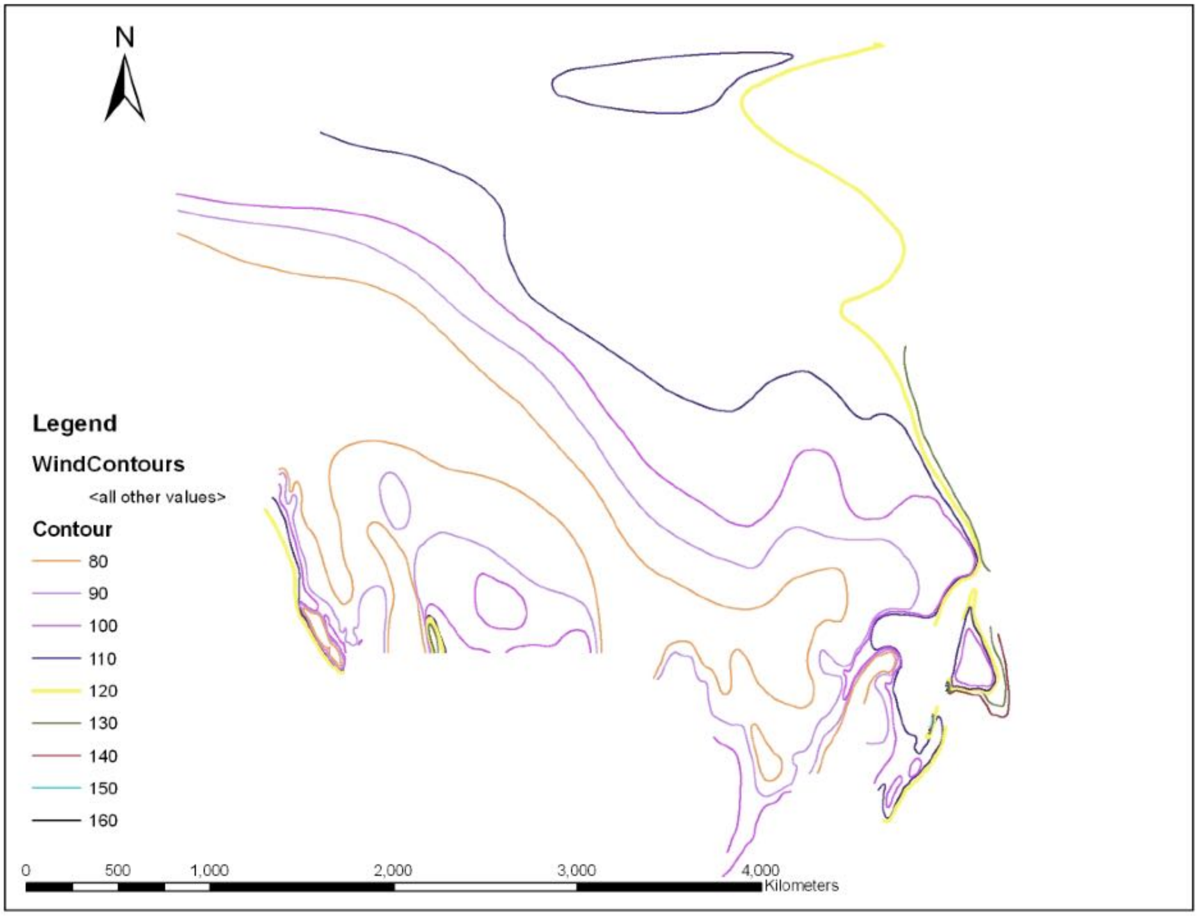

Figure A-1: Canada: 50 Year Return Period Wind Map (Contour Lines) (km/hr) 10-minute

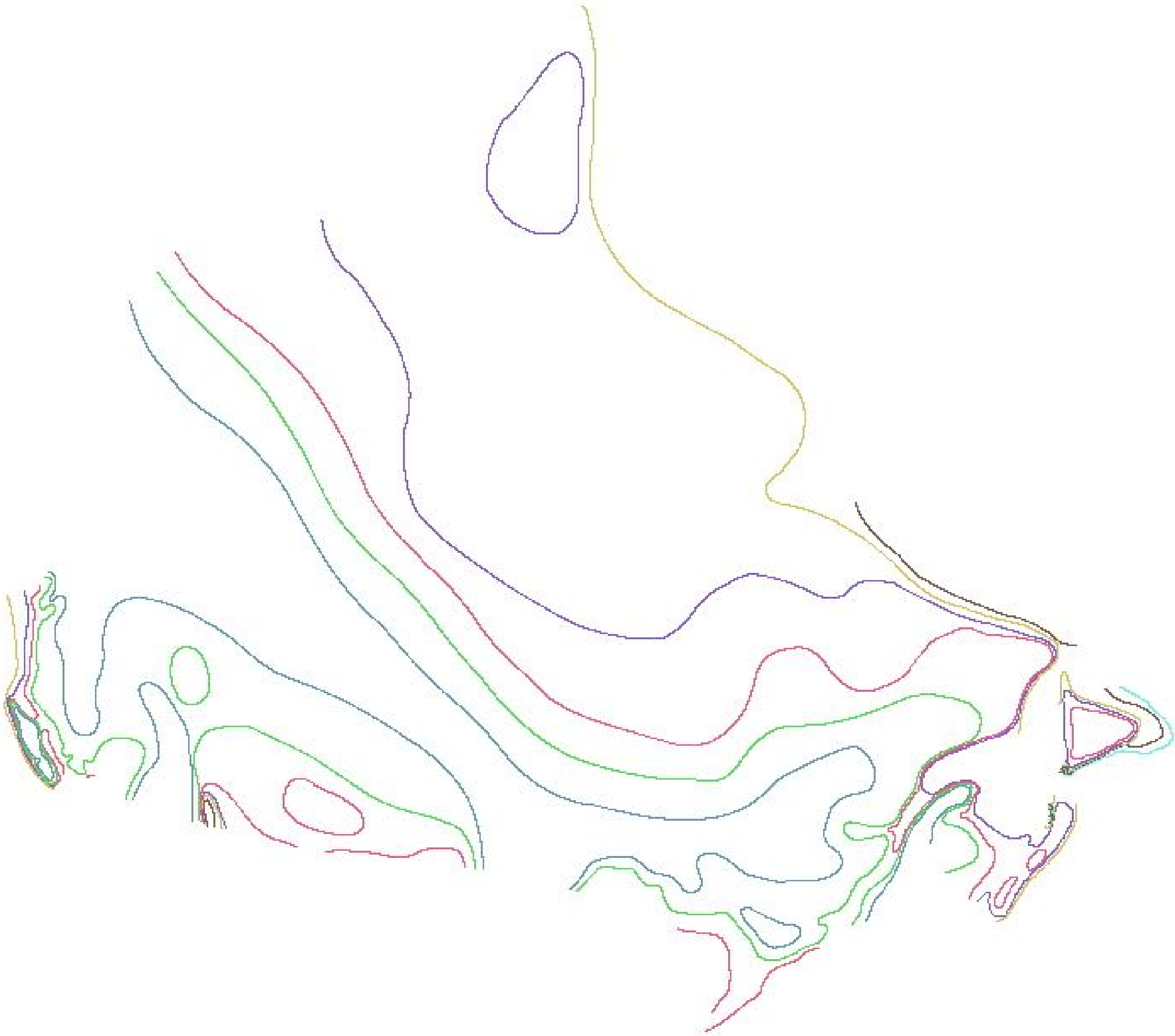

Figure A-2: Canada: 50 Year Return Period Wind Speed (km/hr) (UTM-15N)

A.2 Wind Speed Map - USA

The basic wind speed map for the USA (as of 2005) is shown in Figure A-3. This map is the same as included in ASCE-7-05 (ASCE 7, 2005). It assumes a 50-year return period and represents a 3-second gust.

The reader should be aware that the "stippled" zones in California reflect areas that have higher wind speeds than adjacent areas. It should be recognized that the California wind map in ASCE 7 (including 2005, and updates through 2022) are approximate, and do not reflect recent studies that show detailed wind speed contour speeds, in many locations well in excess of 100 mph. It should be recognized that the wind speed / pressure design provisions of General Order 95 (CPUC) for transmission towers in California (generally specified as 8 pounds per square foot pressure to working stress limits) results in design-level wind speeds of about 56 mph (working stress design), or about 70 mph (limited to steel yield stress or buckling limit with Factor of Safety ~1.0). The reader is encouraged to consider design of transmission towers and distribution feeders in California using latest wind speed contour maps, reflecting latest anemometer data, which will exceed the CPUC 8 psf criteria as well as the wind speed criteria in Figure A-3, in the stippled areas in Figures A-3 and A-4, as well as other areas in California. For example, a wind storm (without precipitation) occurred between November 30 and December 1 2011, with reported gust speeds of 140 mph at ridge tops, that resulted in widespread damage to overhead power lines along the San Gabriel Mountains in Los Angeles County; power outages were estimated at about 500,000,000 customer-minutes, affecting Southern California Edison and Pasadena Water and Power customers.

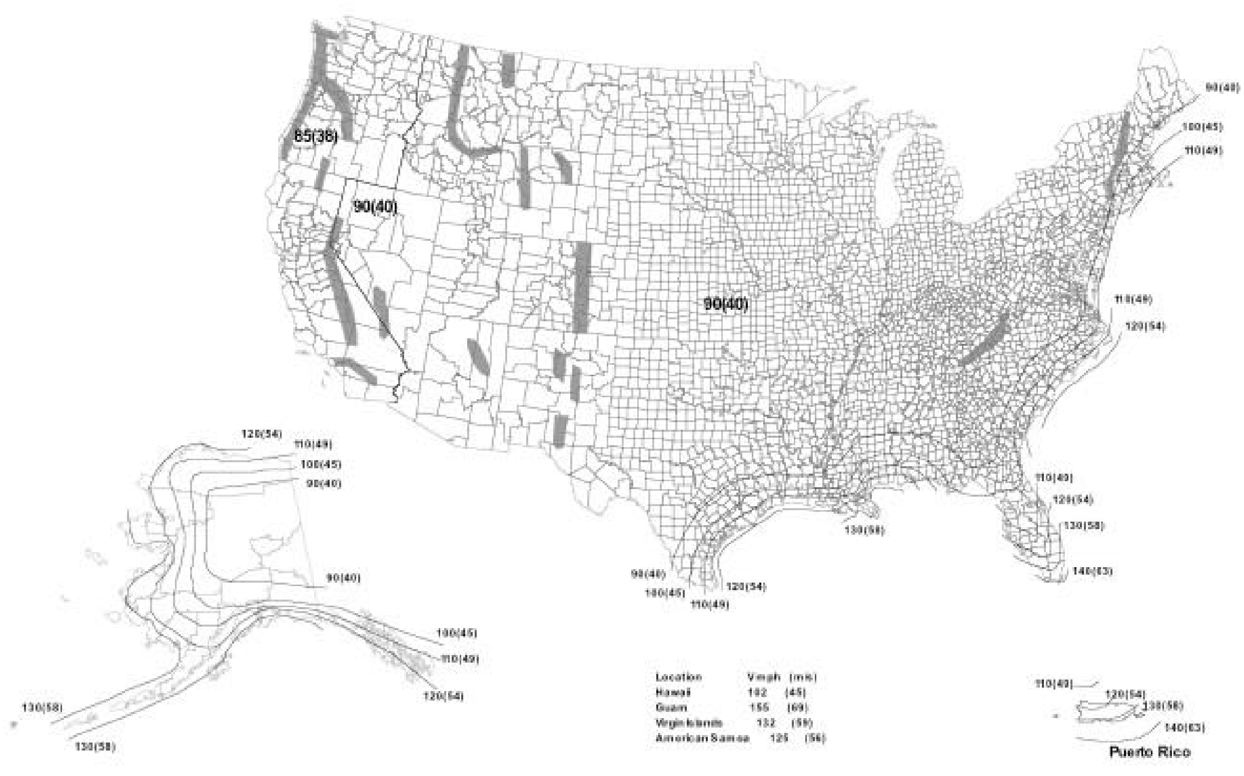

Figure A-3: USA: ASCE 7-05 Wind Map, 50-Year, 3-Sec Gust (mph, m/s)

The development of the USA wind map, Figure A-4, (ASCE 7 2010) varies from that in ASCE 7-2005 (as well as the earlier ASCE 7 1998) with the following major differences:

- New Hurricane Simulation Model that results in changed wind speeds along coastal Florida (see Figure A-5)
- Wind map assumes strength-based design (wind load factor = 1.0 for strength design, = 0.63 for allowable stress design)
- The map corresponds to a 700-year return period for use with Category II structures. The equivalent ASCE 7-2005 map can be derived as (ASCE 7-2010) x 0.791.
- Category III and IV structures use 1,700 year Return Period winds.
- Category I structures use 300 year Return Period winds.
- See ASCE 7-2010 for the definitions of Category I, II, III and IV structures.

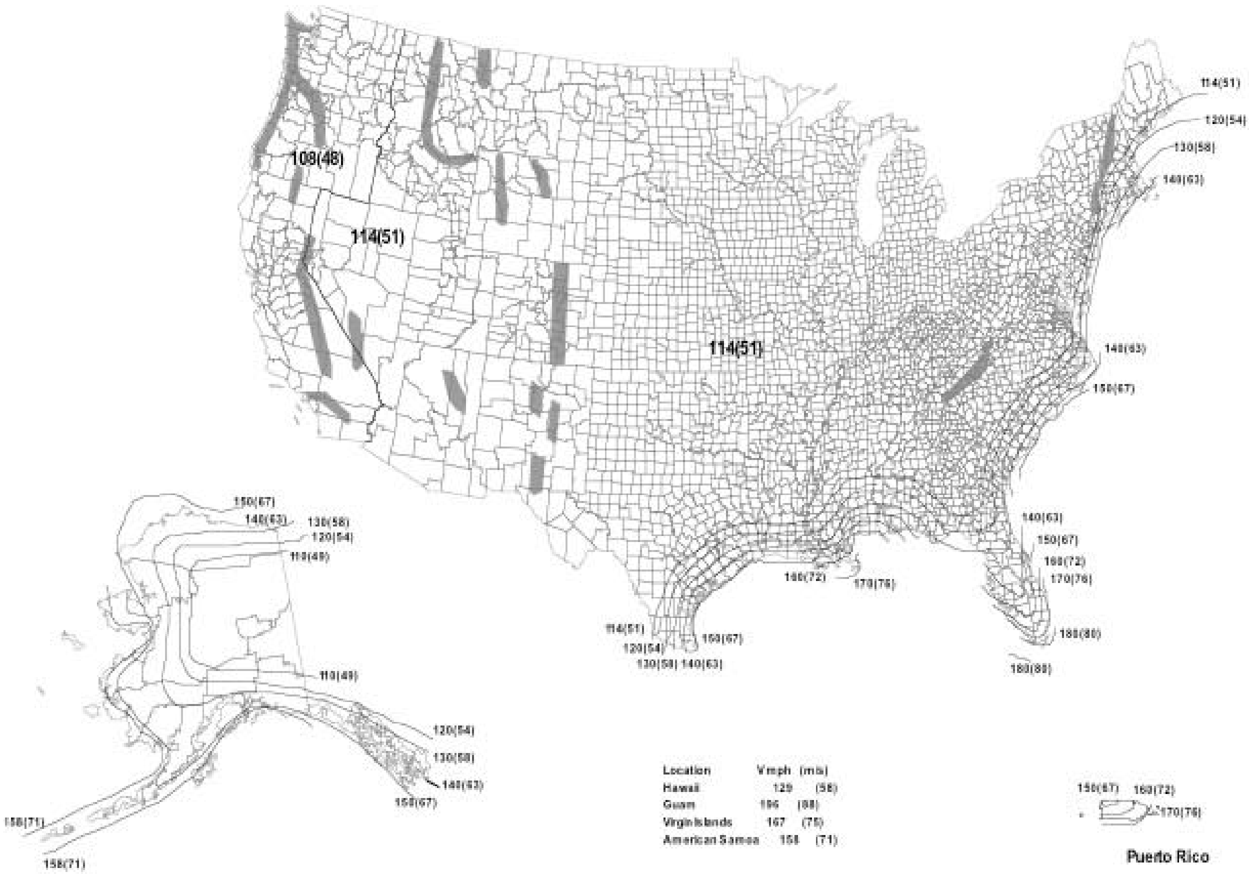

Figure A-4: USA: ASCE 7-10 Wind Map (mph, m/s) 700 Year RP

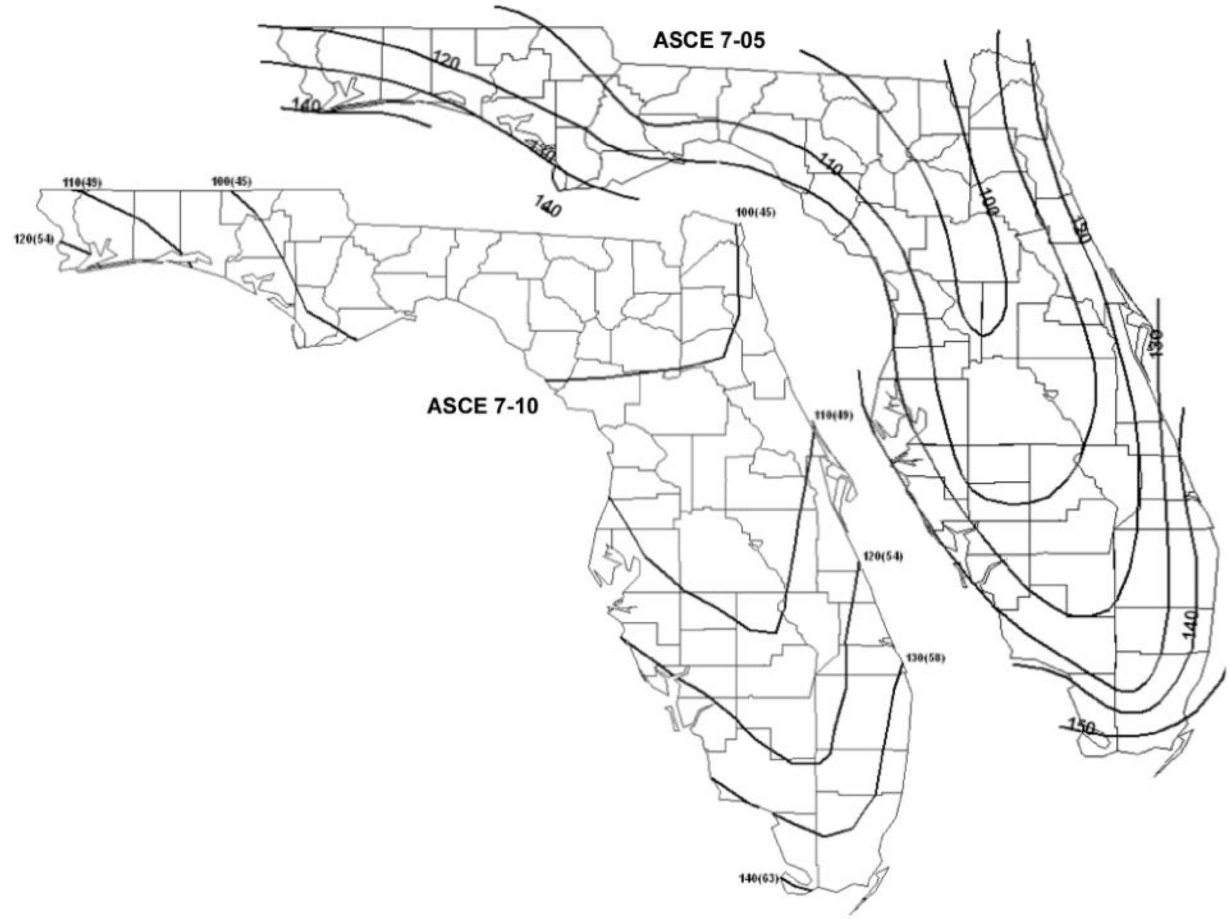

Figure A-5: FLORIDA: ASCE 7-10 and ASCE 7-05 Wind Maps, mph, (m/s) 50 Year RP

A web site maintained by the Applied Technology Council can be used to calculate the wind speeds associated with different return period intervals. As of November 2011, the ATC web site is: http://www.atcouncil.org/windspeed/

A.3 Wind Speed Map – Pacific Northwest (BPA)

Using data from Pacific Northwest weather stations, BPA developed a wind map for its service area, Figure A-6. This map reflects weather station data through 1980. Highest 50-year wind speeds (80 mph) are along the Pacific Coast, along the Columbia River Gorge and locally in Southwest Idaho.

One problem in evaluating transmission towers using probabilistic maps (like Figure A-6) is that a single (scenario) storm will produce varying wind speeds through the course of the storm. For example, Figure A-7 shows the wind speed isotachs for a scenario storm. This map is patterned after the Columbus Day storm that struck the Pacific Northwest in October, 1962. This type of storm is considered to have the potential to cause the highest wind speeds over the greatest area in a single storm, for this part of the USA. This type of extratropical storm[2] forms in the eastern Pacific ocean when warm subtropical air from the southwest meets cold air from the Gulf of Alaska. Such low

[2] Extratropical cyclones are also sometimes called mid-latitude cyclones or wave cyclones. These are synoptic scale lower pressure weather systems that occur in the mid-latitudes having nether tropical or polar characteristics. These storms are commonly called "depressions" or "lows" in common parlance.

pressure systems deepen rapidly as they approach the west coast, then turn sharply northwards at they parallel to the coasts of Oregon and Washington.

A challenge in developing scenario storm is to describe a statistical treatment of high winds for a specific storm, such that the scatter about mean values for various averaging periods is quantified.

The gust factors plotted on the graph are based on 10-minute mean wind speeds at 10 meters over open exposure (Zo = 0.03). Data from The Storm King (Storm King online), the Western Region Climate Center, and other sources were used to extract the highest reported wind speeds during the Columbus Day Storm.

The following procedure was used to develop the scenario storm used in Figure A-7.

1. Convert the wind speeds at each reporting station to the standard 10-meter (33 feet) height. For stations where the height of the anemometer was unknown, it was assumed to be 20 feet.

2. Much of the available wind speed data from the 1962 storm were reported in terms of the 1-minute or fastest-mile. The work of Durst was used to convert the reported data to 10-minute means.

3. Using Masters' gust factor graph, the gust factor for a 5-second duration at a 95% non-exceedance level, is estimated to be 1.8 times the 10-minute means.

4. Increase all the calculated 5-second gusts by 5% because the center of a Type I scenario storm may approach closer to the coast, and/or be a little deeper than was the case for the 1962 Columbus Day storm.

5. Plot the wind speeds on a map of the Northwest and draw the resulting isotachs (mph). Occasionally, there were data points that seemed either too low or too high when compared to the surrounding stations. When these anomalous values could not be explained by nearby terrain features, adjustments were made to better conform to the overall picture.

6. Note that the wind speeds in Idaho and Montana are below 60 mph, which is to emphasize that this map is *not* a return period wind map.

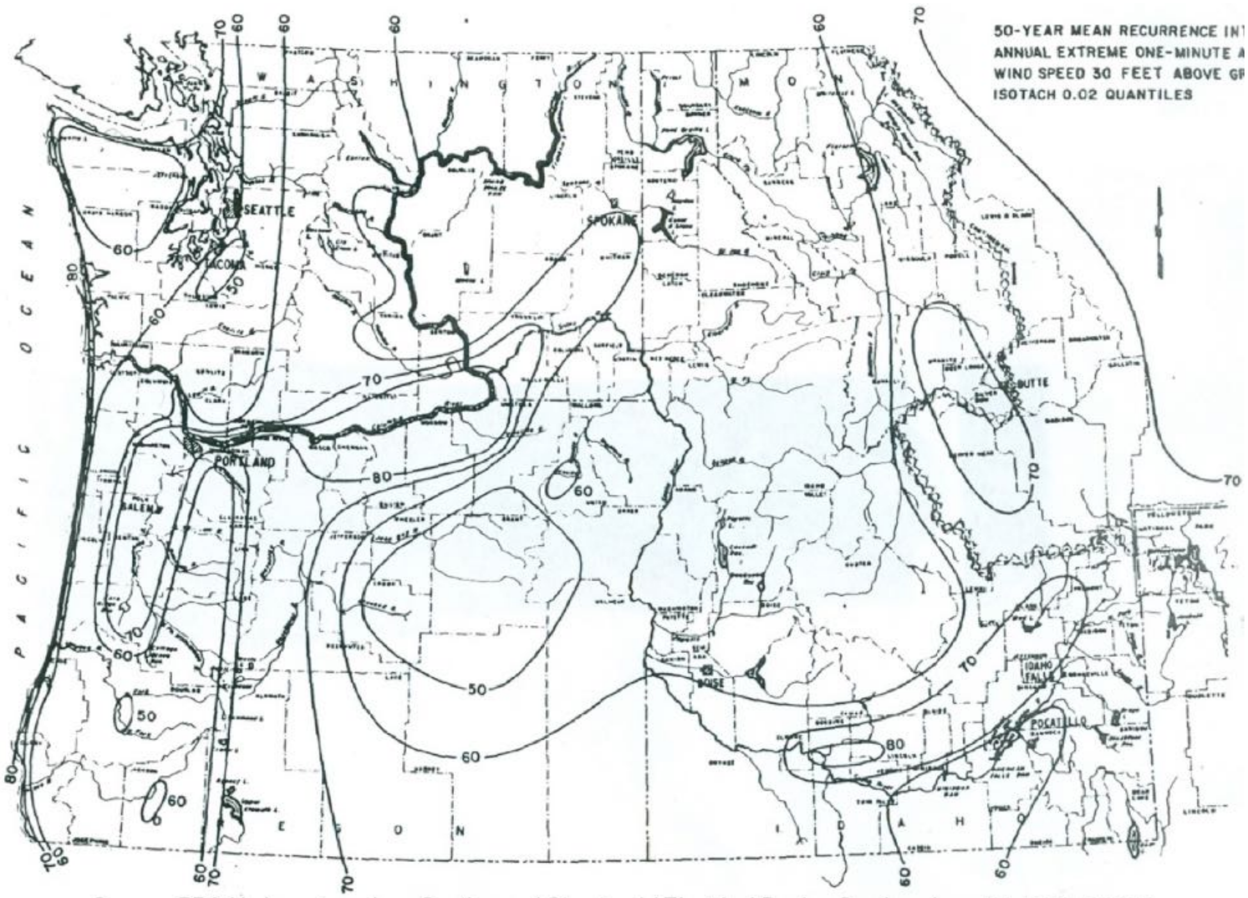

Figure A-6: PNW: BPA Wind Map, 1980 (50 Yr, mph, at height Z = 30 feet, One Minute)

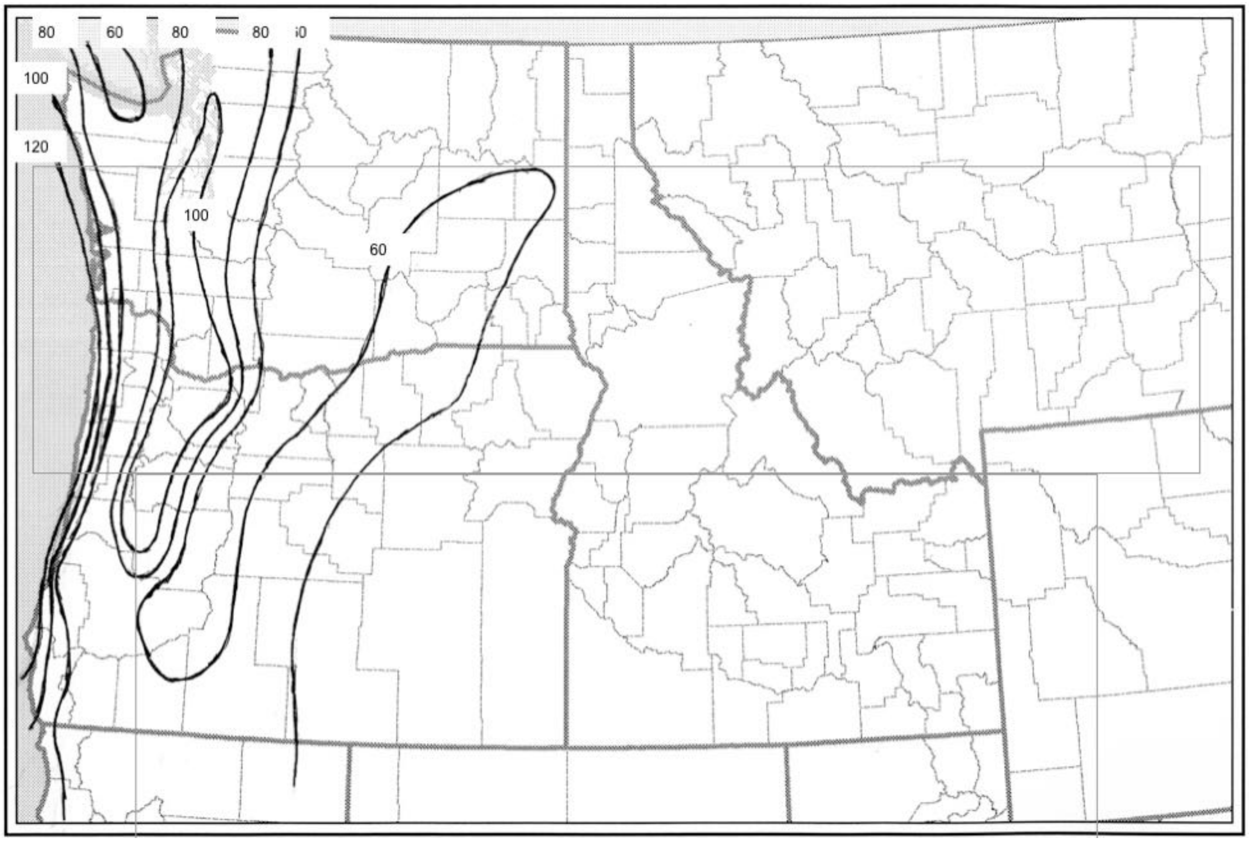

Figure A-7: USA: SCENARIO STORM, MPH, 5-sec Gust, Z = 10 m, 95% non-exceedance

The wind speeds in Figure A-7 can be adjusted from the 95% non-exceedance to the 50% non-exceedance interval using a gust factor adjustment, as follows:

$$Gust\ \ Factor = \mathrm{mode} - \frac{\ln(-\ln(p))}{\alpha}$$

$$\mathrm{mode} = -0.0493 * \log_{10}(t^2) - 0.0601 * \log_{10}(t) + 1.5309$$

$$\alpha = 4.7252 * \log_{10}(t^2) - 3.6592 * \log_{10}(t) + 14.775$$

p = exceedance percentile between 0 and 1

t = gust duration

For example, assuming a 3-second wind speed and a 95% non-exceedance probability, the gust factor is 1.624 (alpha = 17.538; mode = 1.455, GF = 1.624). Similarly, assuming a 50% non-exceedance probability, the gust factor is 1.476. Then, to convert from Figure A-7 (95%, 5-second wind) to 50%, 3-second wind, multiply the values in Figure A-7 by 1.476 / 1.624 = 0.91.

A.4 Hurricane Winds for the Gulf and Atlantic Coasts (USA)

The National Bureau of Standards (NBS, 1980) developed a database of wind speeds for hurricanes along the Gulf and Atlantic coasts of the USA. The results are shown in Figures A-8, A-9 and A-10.

The wind speeds are in mph, and are sustained wind speeds during hurricanes. The return periods are 10, 25, 50, 100 and 2,000 years.

The NBS also developed a database that provides hurricane wind speeds for various locations (counties) in the USA. The database shows "zero" wind-speed values for some counties more than 200 km inland from the coast for 10- 25- and sometimes 50-year intervals; such "zero" entries indicate that hurricanes along the nearby coasts are either too rare or too small (category 0, 1) to induce any wind at those locations.

The wind speeds in Figure A-9 factor in the slowdown of wind speed from the coast to inland locations, and already factors in the effects of surface roughness.

Hurricane winds vary substantially depending on location within a hurricane, relative to the eye of the hurricane. In any hurricane, the land area affected by the maximum sustained winds of the storm is a small fraction of the total area affected by the hurricane.

For example, Hurricane Andrew (Figure A-11) was a Class 4 hurricane with maximum sustained winds over land of about 136 mph. The strongest winds of a hurricane are near the outside of the wall of the eye. Strongest winds occur when the speed of movement of the hurricane add to the rotational winds within the hurricane. Hurricane Andrew moved east to west; therefore the highest winds were on the north eye wall where the translational and rotational wind speeds added. On the south eye wall, the translational and rotational winds were opposite and partially cancelled out. Thus, the peak winds on the south eye wall were about 10 mph lower than on the north eye wall.

Figure A-11 shows that the lowest pressures (925 mb) (and thus highest sustained wind speeds from Hurricane Andrew of 130 mph or higher) were experienced only in a narrow swath along the north eye wall; this high wind speed swath was only about 5 miles wide. Maximum sustained wind speeds decreased sharply away from this swath. About 15 miles north of the maximum wind swath, maximum sustained wind speeds were only 80 mph. Further north, the wind speeds were only of tropical storm strength.

Examining Figure A-11 clearly demonstrates that when analyzing a transmission circuit due to a hurricane, that it would not be rational to expose the entire circuit (unless it is short, say under 5 miles in length) to the maximum wind speeds from that single storm. Similarly, it would not be correct to use the 2000 year return period wind speeds for an entire circuit (say 20 miles long), as long return period wind speeds assume the influence of many hurricanes over the same location over that time period. So, for outage planning and reliability analysis for single events, it is important to model the variation of wind speed over the length of the transmission circuit.

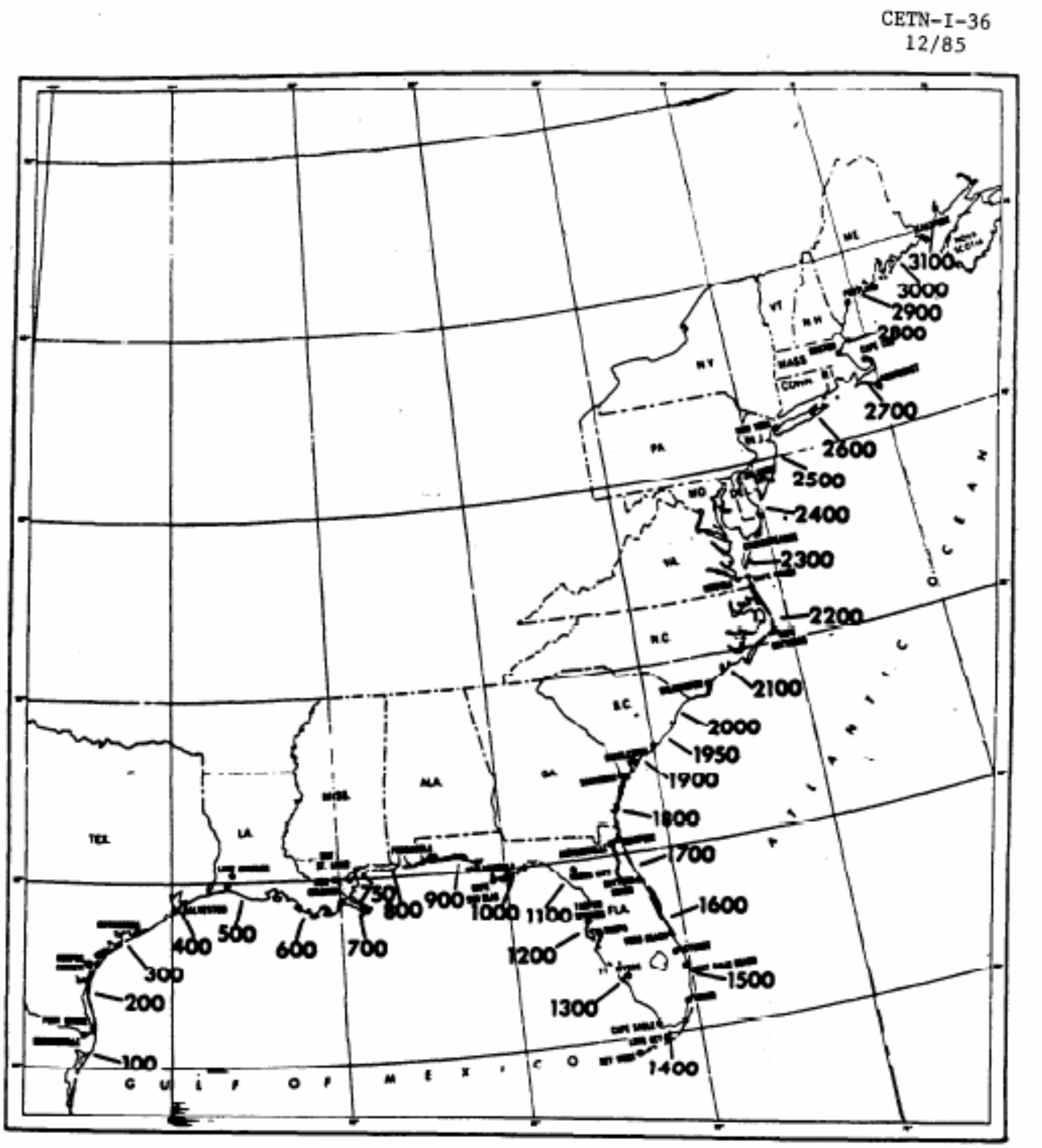

Figure A-8: Hurricane Wind Speeds in the United States (distances in nautical miles)

TABLE 1. ESTIMATED MAXIMUM HURRICANE WIND SPEED (MPH)

RETURN PERIOD

STA	AT THE COAST					AT 200 KM INLAND				
	10	25	50	100	2000	10	25	50	100	2000
150	59	85	98	107	140	39	63	78	89	127
200	59	84	97	106	140	39	63	77	88	124
250	61	84	96	106	138	42	63	75	84	115
300	62	83	95	104	136	46	70	81	91	119
350	63	84	95	104	134	51	72	84	93	121
400	62	81	92	101	133	48	69	81	92	123
450	61	80	91	101	133	46	64	77	89	120
500	59	78	90	100	133	48	66	78	90	119
550	61	80	92	102	132	53	77	88	100	128
600	60	78	90	100	130	59	78	91	100	130
650	61	80	91	100	130	61	80	92	100	130
700	62	81	91	100	132	58	76	86	95	124
750	64	82	92	102	136	53	70	80	89	113
800	65	81	91	101	134	50	66	77	84	110
850	63	80	91	100	136	48	68	78	86	121
900	62	78	88	96	129	49	67	77	85	128
950	61	78	86	93	122	50	67	76	85	128
1000	59	76	84	90	117	47	61	69	76	107
1050	60	76	84	90	115	44	58	66	73	95
1100	61	77	87	93	125	40	56	65	74	102
1150	62	79	89	98	133	46	63	72	84	115
1200	66	84	95	103	138	51	70	81	92	128
1250	73	90	100	108	137	59	80	92	101	136
1300	78	95	104	111	137	66	84	96	104	136
1350	79	96	104	111	139	73	89	100	108	136
1400	80	97	106	113	140	76	92	103	111	139
1450	81	96	105	113	141	77	94	104	113	138
1500	79	93	103	111	139	73	92	101	110	138
1550	74	90	100	109	136	72	91	100	108	136
1600	70	89	99	107	132	64	83	95	105	132
1650	66	85	95	103	126	53	72	84	95	124
1700	62	82	90	98	117	44	61	72	82	116
1750	57	77	85	92	114	36	54	66	74	111
1800	56	76	85	92	117	35	50	62	74	110
1850	56	76	88	98	130	34	50	58	73	103
1900	60	81	93	104	140	40	59	70	82	114
1950	64	87	96	105	139	47	66	77	87	115
2000	64	85	96	106	136	55	75	85	94	121
2050	68	86	96	105	136	58	77	87	95	122
2100	70	84	98	106	136	63	83	93	101	134
2150	71	88	99	107	136	66	85	95	103	136
2200	70	87	97	105	134	69	87	98	105	134
2250	64	81	91	99	132	63	82	92	98	124
2300	58	75	86	95	128	57	74	85	93	121
2350	51	69	81	92	127	47	66	77	87	125
2400	48	65	78	91	125	40	61	77	87	125
2450	47	65	79	91	130	32	54	68	82	129
2500	49	72	87	98	136	31	52	69	83	120
2550	51	77	92	101	138	32	61	76	90	130
2600	53	81	95	104	140	37	70	84	96	133
2650	55	82	95	105	141	42	74	87	98	134
2700	54	80	93	104	139	41	76	89	99	132
2750	50	74	88	99	137	35	74	89	100	133
2800	46	69	82	96	133	26	66	81	92	127
2850	42	62	77	87	132	22	53	71	86	121
2900	39	59	71	85	128	21	46	66	83	117

Figure A-9: Wind Speed Data at the Coast and 200 km Inland

Recurrence Interval (Years)	American Samoa (Pago Pago)	Federated States of Micronesia (Ponape)	Guam (Agana)
10	45	45	100
25	59	59	126
50	69	69	145
100	80	80	184
2000	138	138	190

Recurrence Interval (Years)	Hawaii (Honolulu)	Northern Mariana Islands (Saipan)	Palau (Koror)
10	49	100	70
25	71	127	91
50	87	147	107
100	100	155	125
2000	140	190	190

Recurrence Interval (Years)	Puerto Rico (San Juan)	Republic of Marshall Islands (Majuro)	Virgin Islands (St. Thomas)
10	80	34	80
25	95	39	95
50	104	53	104
100	113	61	113
2000	143	115	143

Sources:

Hawaii	Historic data on hurricanes. Roy T. Matsuda, National Oceanic and Atmospheric Administration, 1994
Pacific Islands	Wind speed and recurrence interval data. Captain John Rupp, Joint Typhoon Warning Center, Guam Naval Facility, 1994
American Samoa	Hurricane risk for this island is approximated as the same as Micronesia.
Virgin Islands	Hurricane risk for these islands is approximated as the same as South Florida.
Puerto Rico	Hurricane risk for this island is approximated as the same as South Florida.

Figure A-10: Hurricane Wind Speeds for Selected Islands and Territories of the USA

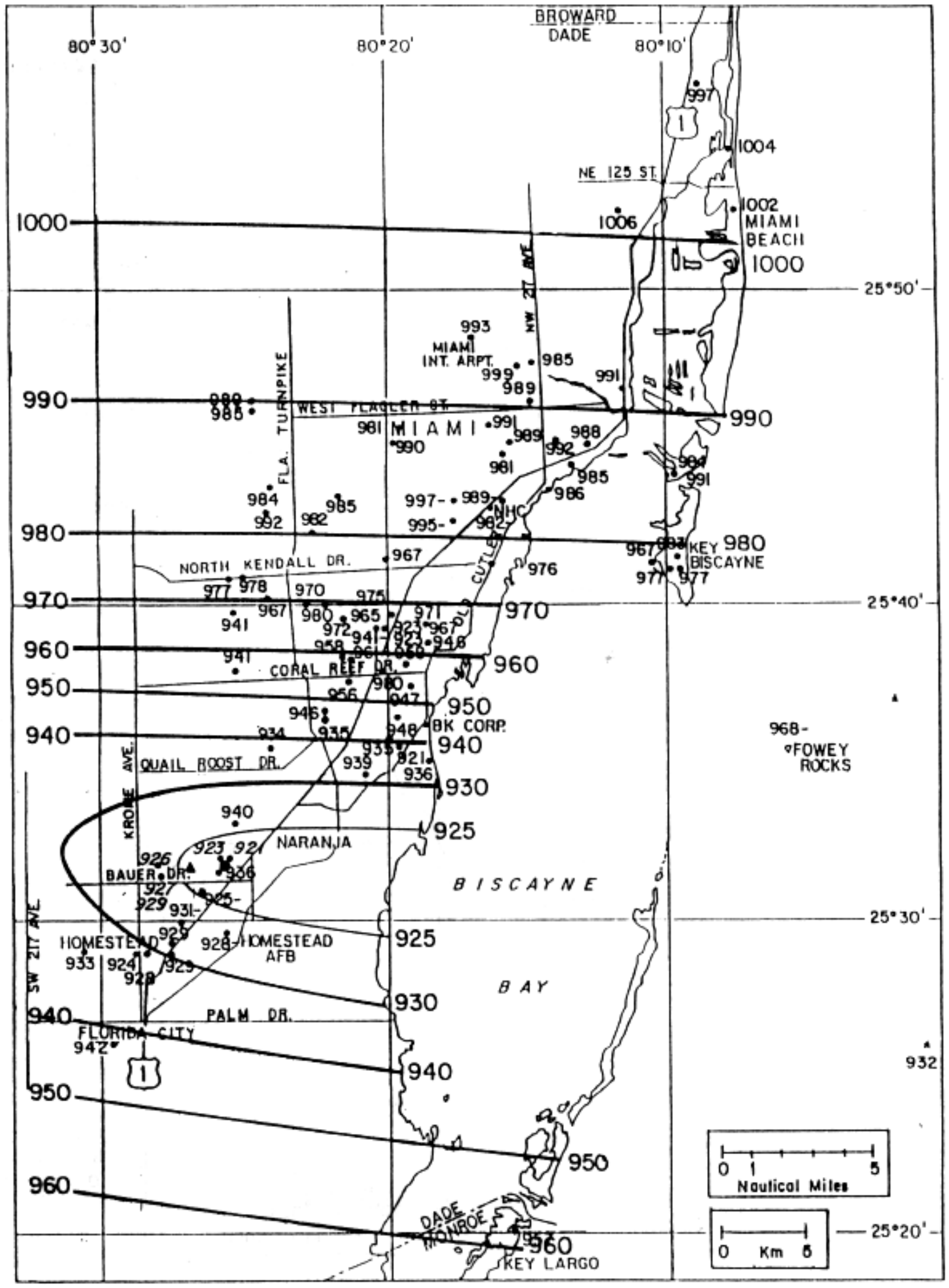

Fig. 4. Observations and smoothed analysis of minimum pressure (mb) during Hurricane Andrew's landfall in Florida. Minus sign indicates that a lower pressure may have occurred. The five italicized labels (near Bauer Dr.) show readings recalibrated and considered reliable in pressure chamber tests. Nearby bigger dot indicates neighborhood of lowest analyzed pressure (922 mb). Offshore reading of 932 mb reported from reconnaissance aircraft on plane's final pass through Andrew before landfall over Florida.

Figure A-11: Geographic Variation of Hurricane Andrew Wind Speeds (Source: Ed Rappaport, National Hurricane Center, 1992)

APPENDIX B. ICE MAPS

Appendix B describes various ice maps that can be used as input to reliability analyses.

B.1 Glaze Ice Maps - Canada

A glaze ice map for Canada is shown below. The contour lines are mapped in Figure B-1 as:

- Projection: Lambert Conformal Conic. False Easting: 0. False Northing: 0. Central Meridian: -96.0. Standard Parallel 1: 50.0. Standard Parallel 2: 70.0. Latitude of Origin: 40.0. GCS North American 1983

Figure B-1 identifies a 15 mm contour line, through central Baffin Island, but this contour line should be 10 mm. The adjusted (corrected) map is drawn in GCS 1983 in Figure B-2.

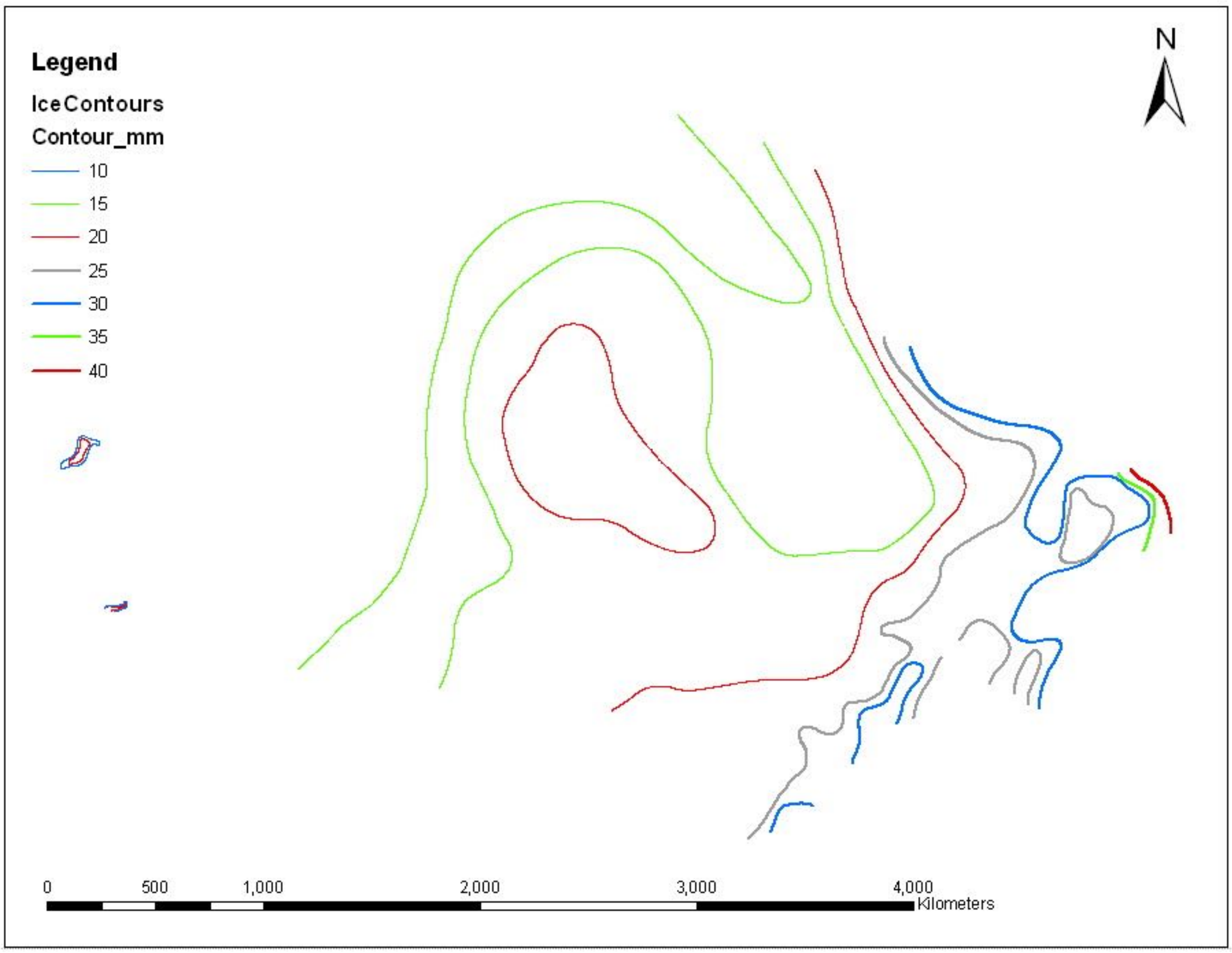

Figure B-1: Canada: 50 Year Return Period Ice Thickness Map (Contour Lines) (mm)

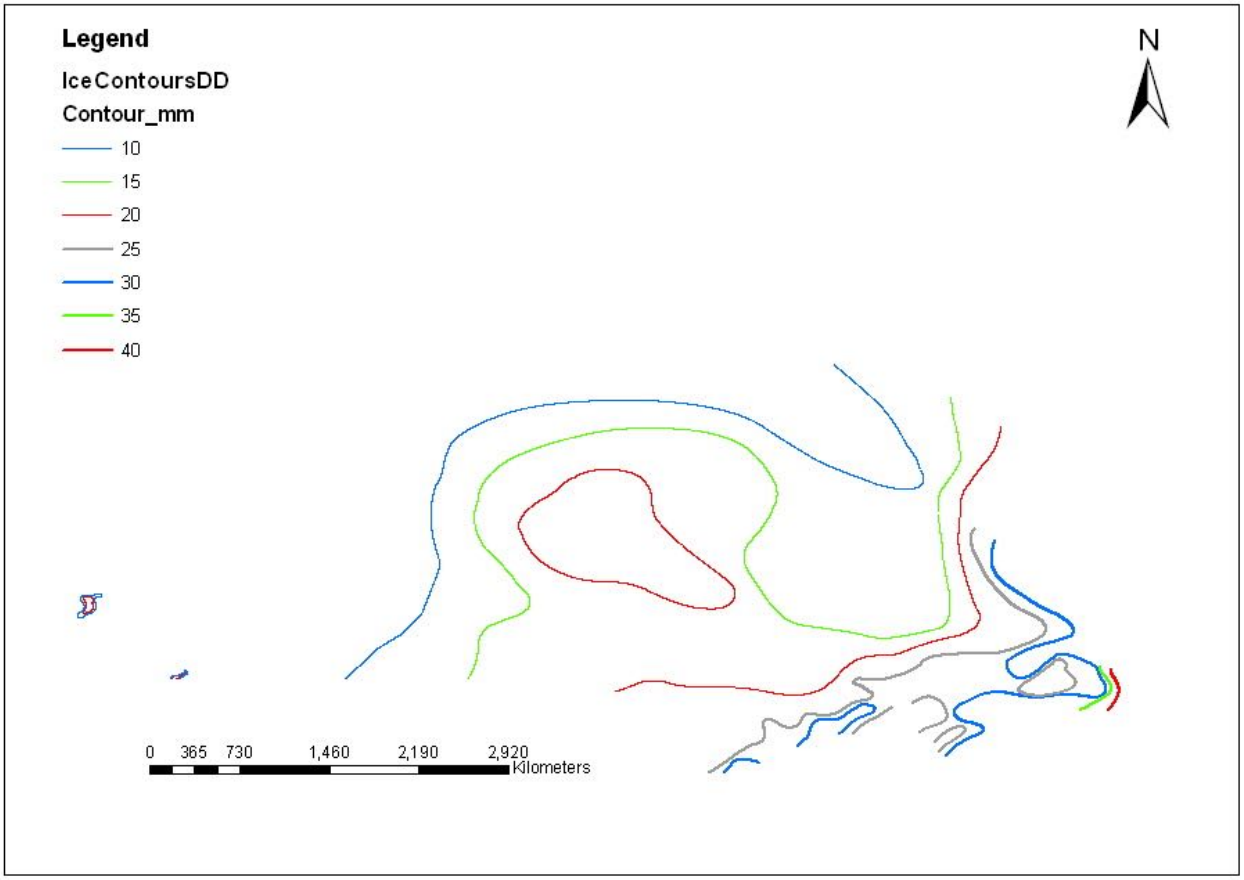

Figure B-2: Canada: 50 Year Return Period Ice Map (Contour Lines) (mm) (Corrected – see text)

B.2 Glaze Ice Maps – USA

Using historical weather station data, Jones prepared the following glaze ice map for the USA (Figure B-3) and locally for the Pacific Northwest (Figures B-4 and B-5).

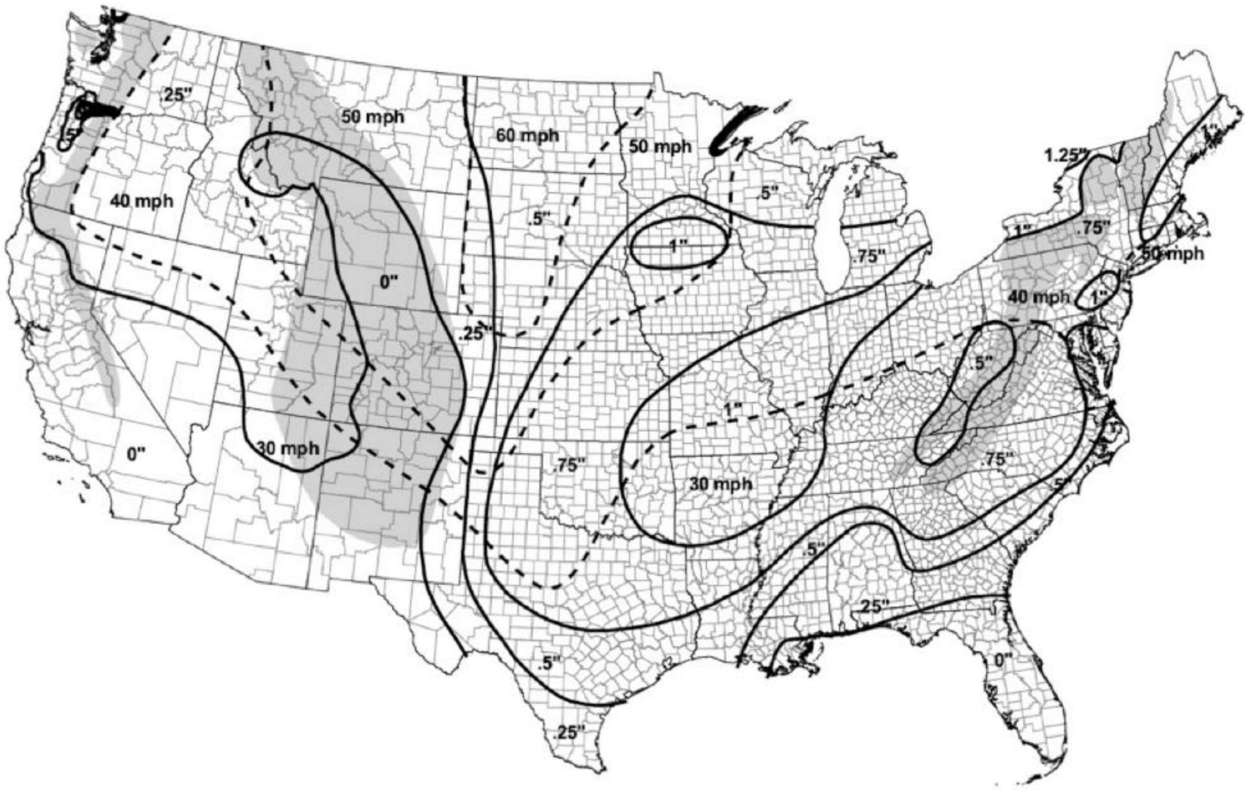

Figure B-3: Glaze Ice Thicknesses with Concurrent 3-Sec Gust, 50-Year (after Jones, 2002)

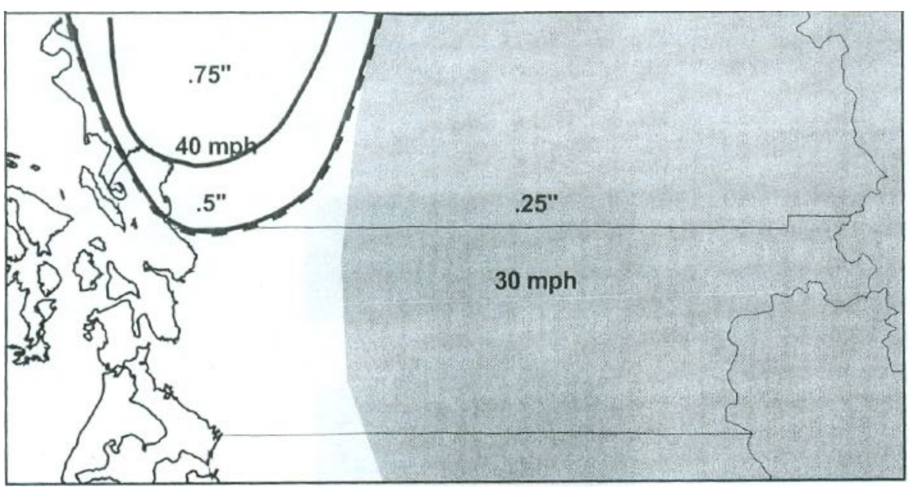

Figure B-4: Fraser Valley: 50 Year Return Period Ice Thickness Map (Contour Lines) (inches) (Jones, 2002)

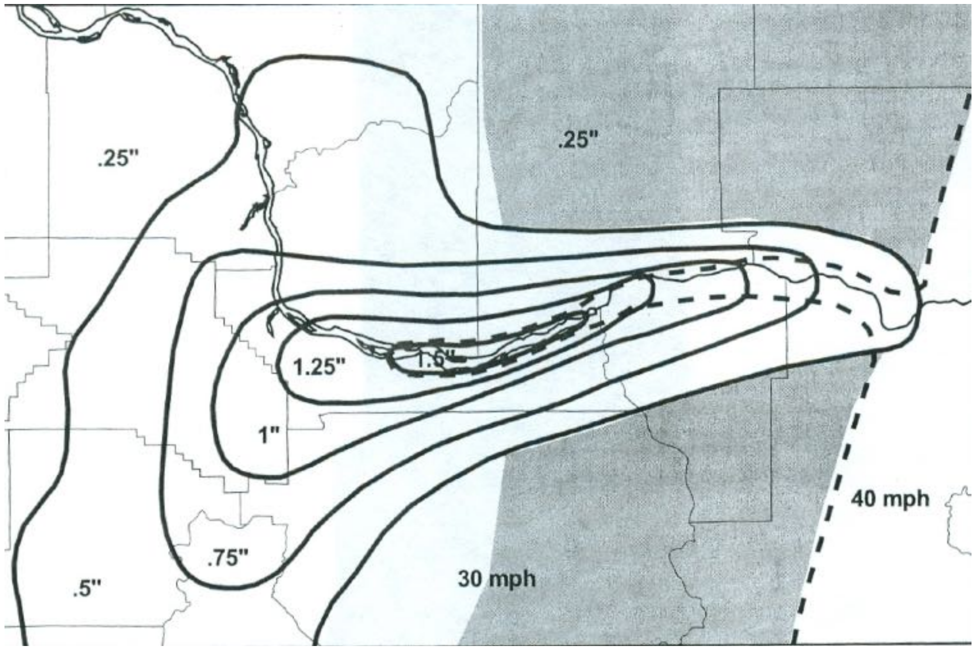

Figure B-5: Columbia River Gorge: 50 Year Return Period Ice Thickness Map (Contour Lines) (inches) (Jones, 2002)

Figures B-4 and B-5 assume that freezing rain is unlikely at elevations over 5,000 feet; and exclude rime ice conditions.

Zone	100 Year	200 Year	400 Year
50-Yr 0.5" and Greater	1.25	1.55	1.91
50-Yr 0.25"	1.34	1.76	2.31
50-Yr 0" zone	1.62	2.42	3.49

Table B-1 Factor to Multiply 50-Year Value

Figures B-4 and B-5 (PNW) assume the following: assume 0.50 inches of radial glaze ice, unless otherwise noted in the map; apply a concurrent 50 mph 3-sec gust. Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain, and in valleys and gorges, may exceed the mapped values.

Figure B-3 (Appalachian Mountains, as indicated by the shaded zone): Ice thicknesses may vary significantly over short distances.

To modify these map for longer return periods, use Table B-1.

The reader should be aware that the provision of concurrent wind speed of 50 mph is based on the historical observations in Canada and USA that the formation of ice (glaze or rime) is consistent with concurrent low wind speeds. However, a ice storm with major rime ice build-up of about 6 inches (radial) occurred in the South Island of New Zealand, followed by a second storm (from the Tasman Sea) with maximum wind speeds on the order of 100 mph: the net result was widespread failure of overhead distribution poles. While the chance of an ice storm immediately followed by a wind storm is small, it can occur, and the reader should recognize that the 50 mph concurrent wind speed provision for the USA may be reasonable, but not necessarily the worst case.

As a general rule of thumb, matched with empirical observations, when glaze ice t is 0.25 inches or more, one can expect significant tree damage. While ice-induced tree damage is not often a factor in causing damage to high voltage transmission lines (owing to wide right-of ways, the height of the towers and assuming removal of tree limbs over the conductors), tree damage is an essential factor in forecasting damage to low voltage (34.5 V and below) power lines that are supported on relatively short (commonly 35 to 60 feet tall) wood (most common) power poles. Any model that is used to forecast damage to low voltage power lines must explicitly include variables that factor in induced damage to trees from ice loading, and actual tree pruning practices along the power line alignment.

This report assumes that for the most part, the failure of transmission towers under ice loading is caused by induced lateral forces on the tower under concurrent wind. In practice, the tower's design must also factor in the dead load capacity to support the weight of the accumulated ice on the conductors, cross arms (often the critical member) and the body of the tower itself. For glaze ice, it is commonly assumed for design purposes that the specific gravity of glaze ice is 0.9 (density of 57 pounds per cubic foot).

Figure B-6 shows a map for the USA and portions of Canada with the footprints of damaging ice storms from 1948 to 2002. Not included in this map are the footprints of ice storms that have affected New Brunswick and Newfoundland, of which there have been many.

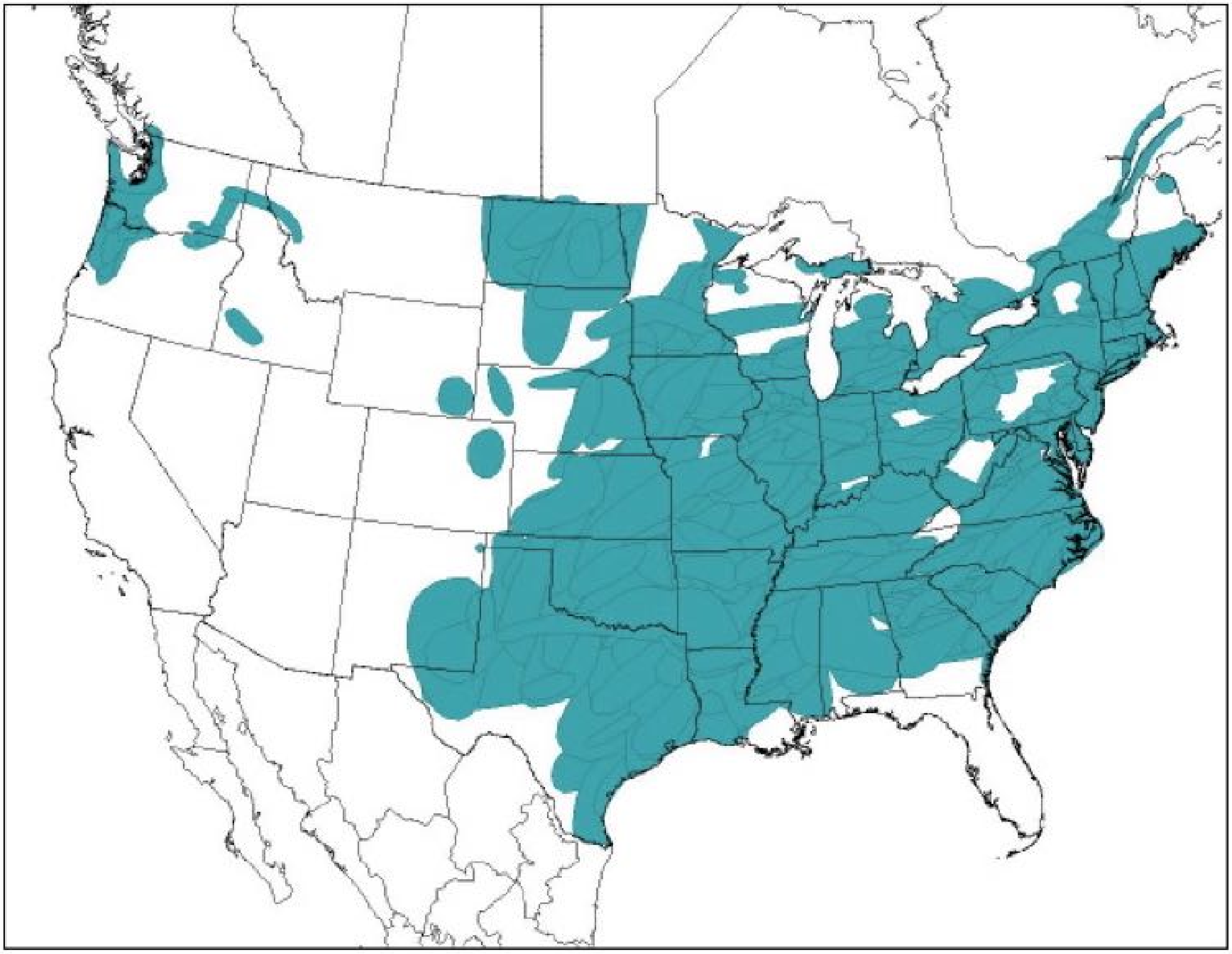

Figure B-6: Footprints of damaging Ice Storms, 1948-2002 (after Jones, 2002)

APPENDIX C. HURRICANES

C.1 Hurricane Return Periods

Many of the wind maps in the USA and Canada are geared towards a basic wind speed set at a 50-year return period. The selection of the 50-year return period is partially based on the fact that most weather stations have had less than 50-years of data from which to make forecasts; and that for most of the two countries, storms happen regularly enough such that a 50-year target wind speed might seem realistic for many purposes. Newer maps for the USA show the basic wind speed at a longer return period interval, such as 1,700 years as mapped in ASCE 7-2010.

When dealing with hurricanes, the use of a 50-year return period is potentially faulty. This is because a weather station along the Gulf coast may not have experienced a hurricane in the past 50 years, so relying directly on a 50-year sample of data would result in a wind speed that is so low as to result in gross failure of structures should they actually be exposed to a hurricane.

From a weather forecasting point of view, there is really no meteorological reason why a hurricane should not make landfall in east Texas versus west Louisiana; we do not have enough physical evidence to show that there is a process whereby the upper level jet stream (steering winds) would favor one landfall location versus the other. Accordingly, code-developers have taken to a deterministic view of hurricane winds speeds for design. Using available statistics of past hurricane landfalls, one can forecast into the future the chance of a hurricane making landfall anywhere along the Gulf or Atlantic coasts over a sufficiently long return period (say 500 years) to ensure of capturing at least one such event, anywhere along the coast. For structures that are sufficiently critical and whose failure would result in hugely negatively consequences (say a nuclear power plant), the design approach would be to assume that the maximum credible hurricane makes landfall at the location worst for the facility, and then design accordingly.

Not wishing to be so severe for regular structures, the IBC code (ASCE 7-2005) limits the planning horizon to 500 years, and in this fashion one does not get the "worst possible" wind speeds at every location along the coast. This is reflected in the ASCE 7-05 map in Figure A-5, whereby the Florida Keys get a 150 mph wind speed, the Gulf Coast between 130 and 140 mph, and the Atlantic coast of Florida between 120 and 130 mph. This variation is largely because the Florida Keys are expected to have more hurricane landfalls over 500 years than the other coast lines; and partly that the Florida Keys have more Category 4 or 5 landfalls than the other locations. However, should a Category 5 hurricane make landfall north of Miami Beach, it will produce nominally the same wind speeds as if it made landfall in the Florida Keys. If one examines the 700-year return period map (Figure A-4), we see that the wind speeds are 180 mph for the Florida Keys, meaning that over that 700-year time period, it is likely that multiple Category 4 or 5 hurricanes will hit that, and the variability of wind speed within each such event suggests that at least one event will produce higher than median wind speeds.

In the zones affected by hurricanes, the strongest winds have a tendency to blow from the sea. Stronger hurricanes tend to have nearly equal speeds from other directions. For this reason, the code requires the designer to assume that the wind comes from any horizontal direction.

- For other areas of the country, the prevailing direction of high speed winds is almost always from one direction. For example, extra tropical storms along the Pacific Coast have highest winds from the southwest or from the west.
- Most transmission towers have been designed assuming that the wind can come from any direction. This will introduce a level of conservatism when the conductor alignment is more nearly parallel to the prevailing high wind direction.
- When examining the reliability of high voltage transmission towers, it might be suitable to consider wind velocity (speed plus direction).

C.2 Hurricane Saffir – Simpson Scale

Category One Hurricane:

Winds 74-95 mph (64-82 kt or 119-153 kph). Storm surge generally 4-5 ft above normal. No real damage to building structures. Damage primarily to unanchored mobile homes, shrubbery, and trees. Some damage to poorly constructed signs. Also, some coastal road flooding and minor pier damage.

Category Two Hurricane:

Winds 96-110 mph (83-95 kt or 154-177 kph). Storm surge generally 6-8 feet above normal. Some roofing material, door, and window damage of buildings. Considerable damage to shrubbery and trees with some trees blown down. Considerable damage to mobile homes, poorly constructed signs, and piers. Coastal and low lying escape routes flood 2-4 hours before arrival of the hurricane center. Small craft in unprotected anchorages break moorings.

Category Three Hurricane:

Winds 111-130 mph (96-113 kt or 178-209 kph). Storm surge generally 9-12 ft above normal. Some structural damage to small residences and utility buildings with a minor amount of curtain wall failures. Damage to shrubbery and trees with foliage blown off trees and large tress blown down. Mobile homes and poorly constructed signs are destroyed. Low-lying escape routes are cut by rising water 3-5 hours before arrival of the hurricane center. Flooding near the coast destroys smaller structures with larger structures damaged by battering of floating debris. Terrain continuously lower than 5 ft above mean sea level may be flooded inland 8 miles (13 km) or more. Evacuation of low-lying residences with several blocks of the shoreline may be required.

Category Four Hurricane:

Winds 131-155 mph (114-135 kt or 210-249 kph). Storm surge generally 13-18 ft above normal. More extensive curtain wall failures with some complete roof structure failures on small residences. Shrubs, trees, and all signs are blown down. Complete destruction of mobile homes. Extensive damage to doors and windows. Low-lying escape routes may be cut by rising water 3-5 hours before arrival of the hurricane center. Major damage to lower floors of structures near the shore. Terrain lower than 10 ft above sea level may be flooded requiring massive evacuation of residential areas as far inland as 6 miles (10 km).

Category Five Hurricane:

Winds greater than 155 mph (135 kt or 249 kph). Storm surge generally greater than 18 ft above normal. Complete roof failure on many residences and industrial buildings. Some complete building failures with small utility buildings blown over or away. All shrubs, trees, and signs blown down. Complete destruction of mobile homes. Severe and extensive window and door damage. Low-lying escape routes are cut by rising water 3-5 hours before arrival of the hurricane center. Major damage to lower floors of all structures located less than 15 ft above sea level and within 500 yards of the shoreline. Massive evacuation of residential areas on low ground within 5-10 miles (8-16 km) of the shoreline may be required.

APPENDIX D. TORNADOES

D.1 Tornado Design Considerations

Tornado-level winds are specifically excluded in the USA wind speed maps (ASCE 7-2010 and its predecessors). In part, this is because even in the regions of the USA most likely to have tornados, the mean return interval at a specific site is on the order of 400 to 500 years, well outside the planning horizon of most common structures.

However, in consideration of a power transmission line that crosses an entire state, it is almost inevitable that tornado-level winds will impact at least one or more transmission lines (loads on the conductors) and occasionally the tower itself. Because of the tight diameter of tornado wind fields, the common assumption of taking 100% (spans up to 900 feet) or 70% (spans over 900 feet) (BPA practice) of the total conductor span length as experiencing a uniform pressure is not applicable under most tornado conditions.

ASCE (1974) provides a US-map that shows the frequency of tornados for every county in the USA.

D.2 Tornado Database

A tornado storm database has been developed that includes 55,439 records of tornadoes in the USA for the period between 1950 and 2010. The underlying data was developed from the Severe Weather Database as compiled in Storm Data. The database also includes 266,869 records for hail reports (since 1955) and 299,817 records for damaging wind reports (including the 1962 Columbus Day Storm).

For each tornado record, the following attributes are provided, along with interpretation for purposes of evaluation for transmission lines:

- Date, time and time zone.
- State (by two letter abbreviation and by FIPS code).
- Fujita scale (Enhanced Fujita scale after January 2007).
- Number of reported injuries and fatalities.
- Property losses (1955 onwards). Crop losses (2007 onwards). Through 1995, the losses are reported in 9 ranges. From 1996, the losses are reported in millions of $US.
- Starting and ending latitude / longitude pairs for the tornado. This is a simplification, as a single tornado does not have to move in straight paths, and can touch down at multiple locations. However, should one want to do a query of the database with an inventory of transmission circuits, this is a reasonable way to see which (if any) tornadoes have historically crossed the path of the circuit.
- Length of the tornado (in miles) and width (on the ground) of the tornado (in yards). The width of the tornado is based on empirical observations, subject to judgment. The width of the tornado will vary over its length, and the database values for width are thought to be for the widest (or nearly so) along the length.
- The database also includes fields to track the number of states (1, 2 or 3) and counties (1, 2, 3 or 4) that the tornado crossed.

Using this database, along with the locations of circuits and towers, one can do an historical analysis to see the number of transmission towers that have been subjected to tornado loads, over the ~60-year period from ~1950 to 2010.

The database includes maps (when internet access is available) showing the path of the tornado (using starting and ending latitude / longitude pairs). The user can zoom in and out on the maps; the maps can be shown using terrain, satellite images, or road networks as backgrounds. The underlying maps are provided from Google, and are subject to Google's terms of use (which can change at any time).

The database allows the user to enter a photo for each specific tornado, as well as a comment field that the user can enter as text description.

D.3 Tornado Scales

D.3.1 Fujita Scale

Dr. Ted Fujita (1971) developed the Fujita scale to provide a method to rate the intensity of tornadoes. The Fujita scale has limitations such as: lack of precise damage indicators, no account of construction quality and variability; no definitive correlation between wind speed and damage. Where listed below, the wind speeds are fastest one-quarter mile.

F-Scale Intensity Wind Speed (mph) Typical Damage

F0 Gale Tornado 40 – 72 mph. Tree branches broken; chimneys damaged; shallow-rooted trees pushed over; sign boards damaged or destroyed; outbuildings and sheds destroyed. Figure D-1.

F1 Moderate 73 – 112 mph. Roof surfaces peeled off; mobile homes pushed off foundations or overturned; moving autos pushed off the roads; garages may be destroyed. Figure D-2.

F2 Significant 113 – 157 mph. Roofs blown off frame houses; mobile homes rolled and/or destroyed; train boxcars pushed over; large trees snapped or uprooted; air born debris can cause damage. Figure D-3.

F3 Severe 158 – 206 mph. Roofs and walls torn off well-constructed houses; trains overturned; large trees uprooted; can knock down entire forest of trees. Figure D-4.

F4 Devastating 207 – 260 mph. Well-constructed frame houses leveled; structures with weak foundations blown off some distance; automobiles thrown; large air born objects can cause significant damage. Figure D-5.

F5 Incredible 261 – 318 mph. Brick, stone and cinderblock buildings destroyed; most debris is carried away by tornadic winds; large and heavy objects can be hurled in excess of 100 meters; trees debarked; asphalt peeled off of roads; steel reinforced concrete structures badly damaged. Figures D-6, D-7.

F6 Inconceivable 319 – 379 mph. These winds are very unlikely. The small area of damage they might produce would probably not be recognizable along with the damage produced by F4 and F5 wind speeds that would surround the F6 winds.

The following photos show damage from various size tornados. All photos are from Brian Smith, Meteorologist, National Weather Service, Valley NE.

Figure D-1. Typical F0 Tornado Damage. Note the trees are stripped of leaves, but the trees remain standing. Only light roof damage and a few missing shingles.

Figure D-2. Typical F1 Tornado Damage. Note the uprooted trees and missing shingles from the roof. There is significant roof damage.

Figure D-3. Typical F2 Tornado Damage. This home is missing its entire roof but the exterior walls remain intact. Some of the stronger hardwood trees remain standing.

Figure D-4. Typical F3 Tornado Damage. This home is missing the entire roof as well as some of the exterior walls. Trees are blown over or snapped near the base and outbuildings are destroyed.

Figure D-5. Typical F4 Tornado Damage. This home is almost completely obliterated, with no walls standing. The debris from the home is where the house once stood.

Figure D-6. Typical F5 Tornado Damage (Asphalt). The asphalt surface has been peeled off of this road.

Figure D-7. Typical F5 Tornado Damage (Homes). These homes have been completely removed from their original locations. The debris field has been scattered some distance from their foundation.

D.3.2 Enhanced Fujita Scale (EF-Scale)

An enhanced Fujita scale has been proposed (Texas Tech University, 2006). The EF scale tries to rectify known issues with the original Fujita scale. For example, it does not take 260 mph wind speed to completely destroy a well-constructed house and blow away the debris; in fact, this damage state occurs at significantly lower speeds.

Recognizing these limitations, a panel of 23 people attended a meeting in 2001 to try to improve (enhance) the correlation between observed damage with the actual wind speed from tornadoes.

This group developed wind speed indicators based on observed damage to buildings, structures and trees. The group tried to eliminate common building damage due to poor construction quality from being used to overestimate true wind speed. For example, common building construction deficiencies can lead to a "weak link" in the wind load path, which might include: inadequate nailing of roof decking, marginal anchor of roof structure to top of wall, discontinuity of walls between first and second floors, use of cut nails instead of anchor bolts to attach the sill plate to the foundation. The panel of experts estimated the wind speeds to reach various damage states for a number of types of structures that are of interest to high voltage transmission tower operators:

- Trees: hardwood (with regard to secondary impacts)
- Trees: softwood (with regard to secondary impacts)
- Free standing Light Poles, Luminary Poles, Flag Poles
- Free Standing Towers
- Transmission Line Towers
- 23 styles of buildings, ranging from single family residences to warehouses.

To preserve the large database of F0 to F5 (original Fujita scale) observations, the EF scale panel of experts derived the following correlation:

$$Wind_{EFScale} = 0.6246 Wind_{FScale} + 36.393, \ R^2 = 0.9118$$

Where the wind speeds (both old and newer EF Scale) are in mph, 3-second gust. For example, a wind speed forecast to be 200 mph per the original Fujita scale is estimated at 161 mph using the updated EF Scale. Note: to convert from fastest quarter mile speed to 3-second gust, see (Durst, 1960).

Fujita Scale			EF Scale	
Fujita Scale	Fastest ¼ mile Wind speed, mph	3-second gust speed, mph	EF Scale	3-second gust speed, mph
F0	40-72	45-78	EF0	65-85
F1	73-112	79-117	EF1	86-109
F2	113-157	118-161	EF2	110-137
F3	158-207	162-209	EF3	138-167
F4	208-260	210-261	EF4	168-199
F5	261-318	262-317	EF5	200-234

Table D-1. EF Scale Wind Speed Ranges Derived from Fujita Scale Wind Speed Ranges

APPENDIX E. EXPERT-BASED FRAGILTY MODELS

Tables E-1 to E-6 present the fragility models based on expert opinion (Texas Tech, 2006) for the capacity of transmission towers and related facilities due to high wind speeds.

Damage State #	Description	Mean	Lower Bound	Upper Bound
1	Threshold of visible damage	83	70	98
2	Broken wood cross member	99	80	114
3	Wood poles leaning	108	85	130
4	Broken wood poles	118	98	142
5	Broken or bent steel or concrete poles	138	115	149
6	Collapsed metal truss towers	141	116	165

Table E-1. Fragility Model, Power Transmission Towers / Poles (3-second gust, mph)

Damage State #	Description	Mean	Lower Bound	Upper Bound
1	Threshold of visible damage	92	76	113
2	Collapsed cell-phone tower or pole	133	113	157
3	Collapsed microwave tower	136	116	160

Table E-2. Fragility Model, Cell Phone / Microwave Towers (3-second gust, mph)

Damage State #	Description	Mean	Lower Bound	Upper Bound
1	Threshold of visible damage	81	67	100
2	Bent pole	102	85	120
3	Collapsed pole	118	99	138

Table E-3. Fragility Model, Free-Standing Light Poles (3-second gust, mph)

Damage State #	Description	Mean	Lower Bound	Upper Bound
1	Small limbs broken (up to 1" diameter)	60	48	72
2	Large branches broken (1" to 3" diameter)	74	61	88
3	Trees uprooted	94	76	118
4	Trunks snapped	107	93	134
5	Trees debarked with only stubs of largest branches remaining	143	123	167

Table E-4. Fragility Model, Hardwood Trees (Oak, Maple, Birch, Ash) (3-second gust, mph)

Damage State #	Description	Mean	Lower Bound	Upper Bound
1	Small limbs broken (up to 1" diameter)	60	48	72
2	Large branches broken (1" to 3" diameter)	75	62	88
3	Trees uprooted	87	73	113
4	Trunks snapped	104	88	128
5	Trees debarked with only stubs of largest branches remaining	131	112	153

Table E-5. Fragility Model, Softwood Trees (Pine, Spruce, Fir, Hemlock, Cedar, Redwood, Cypress) (3-second gust, mph)

For Table E-1, the raw data from six experts was as follows:

DS #	Expert 1	Expert 2	Expert 3	Expert 4	Expert 5	Expert 6	Mean	Std Dev
1	65	80	85	90	100	80	83.33	11.69
2	80	100	110	100	105	100	99.17	10.21
3	80	100	120	110	110	125	107.50	16.05
4	100	120	115	130	120	125	118.33	10.33
5	120	140	130	150	130	155	137.50	13.32
6	120	130	140	170	130	155	140.83	18.55

Table E-6. Fragility Raw Data, Power Transmission Towers / Poles (3-second gust, mph)

In considering the fragilities in Table E-1, one must factor in the following:

- The six experts were Greg Forbes (meteorologist, the Weather Channel), Don Burgess (Meteorologist, National Severe Storms Laboratory), Doug Smith (Engineer, Wind Science and Engineering center, Texas Tech University), Tim Reinhold (Engineer, Clemson University), Tom Smith, Architect, private consulting practice), Tim Marshall (Meteorologist / Engineer, Haag Engineers).

- None of these six experts are thought to be experts in the design of high voltage transmission towers. Their judgment is based on observed damage to a variety of towers, most likely located exposed to hurricane and some to tornadoes; possibly with no observations of damaged towers in Newfoundland or northwest USA.

Table E-1 DS #6 (collapse of steel lattice tower at 141 mph, with details in Table E-6) makes no distinction between towers with one or two circuits; with or without guy wires; suspension or dead-end type towers; variance in tower design standards in use at time of original construction. Should the user select to use this fragility, it is recommended that unless calibrated to the actual towers the user intends to analyze, this selection be limited to towers constructed post-1965 and within 100 miles of the Gulf Coast of the USA; the fragility is NOT suitable for transmission towers built to lower wind speed standards, such as those in most of coastal and central California.