Cascade Failure of Transmission Lines under Downbursts

Ahmed Shehata
The University of Western Ontario

Supervisor
EL Damatty, Ashraf
The University of Western Ontario

Graduate Program in Civil and Environmental Engineering
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Abstract

Downburst belongs to a category of localized windstorm, called High Intensity Wind (HIW) events that happen during thunderstorms. Because of their extensive length, transmission line structures are vulnerable to failure when a downburst happens with their long paths. The failure can be costly since the collapse of one tower often triggers the collapse of many towers along the line. This is one of the major problems facing the electrical utility industry worldwide. While previous research studies have investigated the behavior and failure of a single tower, the research conducted in this thesis is first to consider the analysis and progression of failure of a segment of a transmission line, including multiple towers and the in-between conductors, under downbursts. A unique numerical model is developed in this thesis to achieve this task. This numerical model consists of various components and their formulation and validation are reported in various parts of the thesis.

The model developed to predict the failure of a single tower is first presented in Chapter 2. Once a tower fails at a certain location, a mechanism is formed, and the rotation of the unstable part tends to increase the conductor tension forces, which can bring the tower into a new state of equilibrium. As such, an analytical solution based on three-dimensional catenary theory was developed in Chapter 3 to predict the conductor’s forces with unlevelled ends horizontally and vertically. The comprehensive numerical model that can predict the progress of failures of the entire line is presented in Chapter 4. The model starts by identifying the downburst configuration most critical to the tower. It predicts the failure mode of the tower, including the post failure new equilibrium state and the associated conductor forces transmitted to adjacent towers. The analysis similarly proceed to subsequent towers till covering the entire segment. Case studies involving studying of the failure of a single tower as well as a segment of a line are presented in the thesis together with a parametric study conducted to assess the effect of downburst jet velocity, the span and the insulator length on the cascade failure of a line.
Keywords

Transmission lines, Downbursts, Conductors, Non-linear Finite element analysis, Wind Engineering, Extensible Catenary, Post failure geometry.
Summary for Lay Audience

Cascaded failures of transmission towers presents a critical loading case often disregarded by the relevant codes and design engineers. Research have shown that a limited number of studies was devoted to provide an understanding on the failure of single and multiple transmission towers. Further, information on the cascade failure of transmission lines under High intensity Wind (HIW) was not found in the open literature. The experimental and analytical techniques presented in the few available studies, cannot be extrapolated to provide information on the failure of transmission lines under HIW. Considering the sparsity of information on this matter, and the critical nature of such structures, this thesis discusses the methodology of a novel numerical method to investigate the failure of transmission lines under downbursts. Due to the localized nature of downbursts, a specific location of such windstorms can lead to the initiation of failure of a single tower, which can propagate leading to the cascade failure of a segment of the line. The developed numerical model can predict the failure of a single tower as well as the cascade failure of an entire line under downbursts. In this model, the collapse of one tower can affect the adjacent towers through the unbalanced forces that develop in the conductors. The model can predict the post failure mechanism of a tower, the corresponding geometry of the adjacent conductors and the induced tension forces evaluated using an in-house developed three-dimensional catenary mathematical model.

In this study, a real line was analyzed using this model as a case study. The analysis determined the failure mechanism of the individual towers and predicted how many towers failed during the time-history of the downburst. A parametric study was then conducted by varying the downburst jet velocities, span of the line and the insulator string lengths to study their effects on the cascade failure mechanisms.
Co-Authorship Statement

This thesis has been prepared in accordance with the regulations for an Integrated Article format thesis stipulated by the School of Graduate and Postdoctoral Studies at Western University. Statements of the co-authorship of individual chapters are as follows

Chapter 2: Progressive Failure of Transmission Towers under Downbursts

Numerical simulation and analyses were conducted by A. Sehahta under close supervision of Dr. A. A. El Damatty. A paper co-authored by A. Shehata, and A. A. El Damatty will be submitted to the Journal of Wind and Structures.

Chapter 3: Extensible Catenary Approach in Analyzing Transmission Line’s Conductors under Downbursts

Numerical simulation and analyses were conducted by A. Sehahta under close supervision of Dr. A. A. El Damatty. A paper co-authored by A. Shehata, and A. A. El Damatty was submitted to the Journal of Engineering Structures.

Chapter 4: Numerical model of Cascade Failure of Self-Supported Transmission Lines under Downbursts.

Numerical simulation and analyses were conducted by A. Sehahta under close supervision of Dr. A. A. El Damatty. Drafts of Chapter 3 were written by A. Shehata and modifications were done under supervision of Dr. A. A. El Damatty. A paper co-authored by A. Shehata, and A. A. El Damatty will be submitted to the Journal of Engineering Structures.

Chapter 5: Parametric Study on Cascade Failure of Self-Supported Transmission Lines under Downbursts.

Numerical simulation and analyses were conducted by A. Sehahta under close supervision of Dr. A. A. El Damatty. Drafts of Chapter 4 were written by A. Shehata and modifications were done under supervision of Dr. A. A. El Damatty. A paper co-authored
by A. Shehata, and A. A. El Damatty will be submitted to the Journal of Wind and Structures.
To my beloved parents Alaa El din and Gehan

To my sister Bassant

To my lovely and wonderful wife Perihan

For patience, support, encouragement, and sharing these years of hard work

To my beloved country, Egypt

To my colleagues at Cairo University

To my supervisor, Dr. Ashraf A. El Damatty
For his continuous support and guidance as well as sharing his experience during these years
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I wish to pay my special regards to my beautiful sister Bassant for all the support and prayers she have been providing.

Above all, I would like to thank my wife Perihan for her love and constant support, for all the late nights and early mornings, and for keeping me sane over the past few months. Thank you for being my muse, editor, proofreader, and sounding board. Most of all, thank you for being my best friend. I owe you everything.
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<th>Description</th>
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<tbody>
<tr>
<td>ASOS</td>
<td>Automated Surface Observing System</td>
</tr>
<tr>
<td>CFD</td>
<td>Computational Fluid Dynamics</td>
</tr>
<tr>
<td>CLAWS</td>
<td>Classify and Avoid Wind Shear</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite element modelling</td>
</tr>
<tr>
<td>JAWS</td>
<td>Joint Airport Weather Studies</td>
</tr>
<tr>
<td>MIST</td>
<td>Microburst and Severe Thunderstorm</td>
</tr>
<tr>
<td>NIMROD</td>
<td>Northern Illinois Meteorological Research on Downbursts</td>
</tr>
<tr>
<td>TI</td>
<td>Transmission Line</td>
</tr>
<tr>
<td>UoB-TWS</td>
<td>University of Birmingham Transient Wind Simulator</td>
</tr>
<tr>
<td>UWO</td>
<td>University of Western Ontario</td>
</tr>
<tr>
<td>WindEEE</td>
<td>Wind engineering, energy and environment</td>
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<tr>
<td>WiST</td>
<td>Wind Simulation and Testing</td>
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</table>
## List of Symbols

<table>
<thead>
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<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$\Delta x$</td>
<td>Insulator incremental displacement in the longitudinal direction, $x$.</td>
</tr>
<tr>
<td>$\Delta y$</td>
<td>Insulator incremental displacement in the transverse direction, $y$.</td>
</tr>
<tr>
<td>$\Delta z$</td>
<td>Insulator incremental displacement in the vertical direction, $z$.</td>
</tr>
<tr>
<td>$A_c$</td>
<td>Conductor’s cross section area</td>
</tr>
<tr>
<td>$A_{lx}$</td>
<td>Conductor’s span length</td>
</tr>
<tr>
<td>$C$</td>
<td>Catenary constant</td>
</tr>
<tr>
<td>$C_{rx}$</td>
<td>Conductor’s longitudinal forces</td>
</tr>
<tr>
<td>$C_{ry}$</td>
<td>Conductor’s transverse forces</td>
</tr>
<tr>
<td>$C_{rz}$</td>
<td>Conductor’s vertical forces</td>
</tr>
<tr>
<td>$D_J$</td>
<td>Downburst jet diameter</td>
</tr>
<tr>
<td>$d_x$</td>
<td>Insulator end displacement in the longitudinal direction, $x$.</td>
</tr>
<tr>
<td>$d_y$</td>
<td>Insulator end displacement in the transverse direction, $y$.</td>
</tr>
<tr>
<td>$d_z$</td>
<td>Insulator end displacement in the vertical direction, $z$.</td>
</tr>
<tr>
<td>$E$</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>Conductor’s strains</td>
</tr>
<tr>
<td>$H$</td>
<td>Conductor’s transverse loads</td>
</tr>
<tr>
<td>$h_f$</td>
<td>Intermediate hinge height</td>
</tr>
<tr>
<td>$h_L$</td>
<td>Conductor’s sag</td>
</tr>
</tbody>
</table>
Mdes  Failed segment destabilizing moments
Mst   Failed segment stabilizing moments
R     Distance between the downburst center to the structure of interest center.
Rx    Conductor’s longitudinal reaction
Ry    Conductor’s transverse reaction
Rz    Conductor’s vertical reaction
T     Conductor’s tension
\( t_f \)  Time increment at which an intermediate hinge is formed in the tower’s shaft
Tmn   Conductor’s average tension
To    Conductor’s horizontal tension
\( V_{rd} \) Downburst radial velocity
\( V_{vr} \) Downburst vertical velocity
\( V_J \) Downburst jet velocity
w     Conductor’s weight per unit length
W     Conductor’s vertical loads
\( \theta \) Angle between the vertical plane of the transverse direction and the vertical plan connecting the downburst and the structure of interest centers.
\( \theta_d \) Orientation of the tower’s failed segment in the horizontal plan with the longitudinal axis, y.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\theta_f$</td>
<td>Orientation of the tower’s failed segment with the vertical axis, $z$.</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Ratio between the member axial forces to its axial capacity</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Axial Stresses</td>
</tr>
</tbody>
</table>
Chapter 1

1 Introduction

1.1 General

Electrical power networks are critical infrastructure elements necessary for the sustenance of power distribution in urban environments. A modern-day electrical network is comprised of several elements, of which, transmission towers are one of the primary modes of distribution. In a typical transmission line, conductors are suspended via a series of insulators which are connected to the cross arm of supporting towers as shown in Figure 1-1. The towers can be classified as either self-supported or guyed towers, depending on how they are attached to the ground. Transmission lines (TL) extending for thousands of kilometers to provide power to remotely populated regions are prone to failure under High Intensity Winds (HIW) in the form of downbursts and tornados by virtue of their length. Transmission tower-line system as an integral part of extra-high voltage transmission network, with the unique characteristics of high-rise, long span, and high flexibility, can suffer from structural instability and other catastrophic accidents by the action of HIW events. HIW events are usually associated with thunderstorms in the form of rising air and descending air masses. The updrafts are formed by warm moist air and downdrafts are formed by colder air. Fujita (1985) and (1990) defined a downburst as a mass of cold and moist air that drops suddenly from the thunderstorm cloud base impinging on the ground surface and then transfers horizontally. Downbursts cause localized failure at a certain tower of the transmission line which can lead to the collapse of a segment of the line. Several studies in the open literature reported significant losses and emphasized the importance of considering such events in the design of transmission lines. For instance, Hawes and Dempsey (1993) stated that 90% of the transmission line failures in Australia were induced by downbursts. In southwestern Slovakia, Kanak et al. (2007) studied a downburst event that occurred in 2003 where at least 19 electricity transmission towers collapsed.
Figure 1-1 Lattice Self-Supported Tower (Source https://commons.wikimedia.org/w/index.php?curid=19289599)
In China, Zhang (2006) reported the failures of 18 towers belonging to 500 kV lines and 57 towers belonging to 110 kV lines due to strong wind events such as downbursts, tornadoes and typhoons. Most recently, in September 2016, 23 transmission towers failed during a series of downburst events in South Australia (Australian Wind Alliance, 2016). Also, several failures of transmission line system under downburst events have been reported in Canada such as the failure of 19 transmission towers located near Winnipeg, Manitoba reported by McCarthy and Melsness (1996), and the failure of two guyed towers belonging to Hydro One, Ontario (Failure report, 2006).

When a suspension tower fails due to high wind, it imposes un-balanced loads on the adjacent two structures, thus creating longitudinal and transverse loads. If the adjacent structures cannot withstand these loads, then failure will propagate causing a cascade. Tucker (2007) stated that there are three types of cascades are possible in transmission systems, see Figure 1-1;

1. Transverse: caused by tower failures perpendicular to the line which imposes transverse and longitudinal loads;

2. Vertical: caused by failure of phase support (cross-arm, insulator) in suspension structures, typically dropping one phase to the ground over numerous structures;

3. Longitudinal: caused by structural failure in an element in the plane of the line.

Historically, cascading failures have caused significant economic loss and, in some cases, loss of lives too. The damage of cascading failures can be catastrophic. There have been many cascading tower failures around the world reported by Frandsen and Juul (1976), Oswald et al. (1994), EPRI (1996). In 1966, 167 towers collapsed in a 150-kV coast-to-coast transmission line in Denmark. In 1975, 289 transmission line towers failed (cascaded) in Wisconsin. In 1975, 69 structures cascaded in Indiana. In 1993, 17 towers of a 345-kV double circuit transmission line cascaded in Arlington, Texas. In 1993, 64 mi of 345 kV transmission line and 406 supporting structures were lost during an ice storm. In 1998, Hydro Quebec 735 kV transmission line cascaded during an ice storm; an
example of such an infamous cascade failure occurred which left four million people without power for about a week in Quebec, New Brunswick and Ontario. Recently, in 2007 and 2013 Newfoundland has also seen some prolonged power disruptions in many areas.

![Figure 1-2 Cascade failure types](image)

This thesis focuses mainly on the cascade failure of transmission lines under downbursts. In this chapter, a literature review pertaining to the subject of transmission lines behaviour under downbursts is presented. The review focuses first on the downburst wind field measurements, downburst experimental studies and downburst numerical studies. This is followed by coverage for the studies conducted on the structural response of transmission lines under downbursts. An extensive research program was conducted on this subject by the research group of the author’s supervisor at The University of Western Ontario (UWO). The current thesis builds on this research and expand it. As such, the
progress of research conducted by the research group at UWO is covered. This is followed by presenting the objectives and the scope of the thesis.

1.2 Downburst Wind Field

1.2.1 Downburst Field Measurements

Downburst has a localized nature that is quite different from the large-scale wind, such that its structure, scale, and intensity cannot be easily measured in the field spatially and temporally. Studying downburst characteristics can be conducted by field measurements, numerical modeling or experimentally. Research programs such as the FAA/Lincoln Laboratory Operational Weather Studies (FLOWS) (Wolfson et al. 1985), the Northern Illinois Meteorological Research on Downbursts (NIMROD), the Joint Airport Weather Studies (JAWS) (Fujita 1985), the Classify and Avoid Wind Shear (CLAWS), the Microburst and Severe Thunderstorm (MIST) are essentially meteorological studies oriented towards understanding the formation of microbursts and downbursts. Wilson et al. (1984) used the JAWS project Doppler weather radar data and reported the downburst's horizontal and vertical profiles. Different field measurement studies such as by Choi and Hidayat (2002) and Holmes et al. (2008) discussed various decomposition approaches to extract the mean component of the thunderstorm winds. Later, Lombardo et al. (2014) analyzed the archived data obtained by Automated Surface Observing System ASOS to distinguish a range of downburst thunderstorms and compared it to synoptic wind events. Based on field measurements at various European ports such as Genoa, Savona, La Spezia, Livorno and Bastia, De Gaetano et al. (2014) and Solari et al. (2012, 2015) analyzed more than 90 thunderstorm events under the “Wind and Ports” project. An integrated system was utilized based on extensive in-situ wind monitoring network, numerical simulation of wind fields, statistical analysis of wind climate, and algorithms for wind forecasting. Although field studies can provide the actual velocity measurements, it represents a challenging task due to the uncertainty of the event occurrence location and time (localized effect). More importantly, it does not provide the spatial variation, which is very important in studying long structures such as TIs.
1.2.2 Experimental Studies for Downburst Wind Field

Different approaches were used to simulate the downburst wind field physically inside wind tunnel laboratories. Donaldson and Snedeker (1971), Didden and Ho (1985), Osegura and Bowles (1988), Lundgren et al. (1992), Alahyari and Longmire (1994), Yao and Lundgren (1996), Choi (2000), Wood et al. (2001) and Chay and Letchford (2002) simulated a small scale downburst using a jet flow injected from a circular convergent nozzle and impinging normally onto a flat plate in the laminar boundary layer. These simulations are typically used to characterize the downburst wind field and not the effect of the wind on structures. That is due to the inherent constraint of simulating downburst and the structure at the same length scale where the downburst diameter is an order of magnitude greater than that of the structure. In order to overcome such a challenge, researchers adopted large-scale testing facilities where the effect of the downburst on the structure can be modeled.

Large-scale testing facilities, such as Wind Simulation and Testing (WiST) facility at Iowa State University and Wind Engineering, Energy and Environmental research institute (WindEEE) at the University of Western Ontario (Figure 1-3) allow for characterizing both the downburst wind field and its effect on structures. Sarkar et al. (2006) utilized WiST to model microburst events using an impinging jet and studied their effect on a tall building model. Jesson et al. (2015) utilized the University of Birmingham Transient Wind Simulator (UoB-TWS) to simulate the primary vortex of a thunderstorm downburst and to study the pressure distributions over framed structures. Jubayer et al. (2016) used the WindEEE dome facility located in the University of Western Ontario to study the response of a low rise building subjected to a simulated downburst flow. Elawady et al. (2017) performed experimental downburst simulations at the WindEEE dome on a multi-span transmission line which will be explained later in more details.
1.2.3 Numerical Studies for Downburst Wind Field

Downburst events were modeled numerically using two main approaches; (i) the Impinging jet model and (ii) the Cooling source model as shown in Figure 1-4. The Impinging jet model, as proposed by Fujita (1985), relies on the analogy between an impulsive jet impinging upon a flat surface, in which after the touchdown, a radial strong outflow occurs. The Cooling source model was proposed by Anderson et al. (1992). The model relies on the bouncy forces to form the downdraft. In their study, a negative energy source was placed in the computational domain to cool down the air in the cloud base (higher density) which resulted into a negative buoyancy force that formed the downdraft.

dependent downburst wind field based on the Reynolds Averaged Navier–Stokes (RANS) method. Using a Large Eddy Simulation (LES), Aboshohsa et al. (2015) characterized the downburst field under four different exposures based on an Impinging Jet model. Mason et al. (2009) implemented the cooling source method based on a dry, non-hydrostatic, sub-cloud and axisymmetric model. One year later, Mason et al. (2010) extended this work to a three-dimensional modelling. In both studies, the Scale Adaptive Simulation (SAS) method developed by Menter and Egorov (2005) was used, which is an improvement for the unsteady Reynolds-Averaged Navier–Stokes (URANS) method employed to predict unsteady turbulent flow. However, Gant (2009) reported that the SAS method appears to be over-predicting the turbulent viscosity of jet-type flows. Vermeire et al. (2011) simulated the downburst using the cooling source approach with Large Eddy Simulation (LES) and the results showed a good agreement with those reported by Mason et al. (2009) and a disagreement with the impinging jet models.

![Cooling source model](image1.png) ![Impinging jet model](image2.png)

**Figure 1-4 Numerical model for Downbursts (Ibrahim 2017)**

1.3 Studies on TLs Response to Downbursts

Following the collapse of a number of transmission towers during a downburst event in 1996 at the province of Manitoba, Canada, an extensive research program related to this subject has been conducted at the University of Western Ontario. Shehata et al. (2005) developed a finite element numerical model that simulates transmission line system including the towers, conductors and insulators under downburst loading. The conductors
were studied separately and then their reactions were reversed and applied at the tower connection points. For the downburst loading, a scaling procedure was developed to scale up a small impinging jet wind field developed by Hangan et al. (2003). Based on this basic model, Shehata and El Damatty (2007), Shehata and El Damatty (2008), and Shehata et al. (2008), studied a guyed tower while Darwish and El Damatty (2011) studied the behavior and the failure modes of a self-supported tower. Those studies considered only the main component of the downburst wind field in quasi-static analyses. It should be noted that because of the transient nature of the downbursts, the mean component also varies with time, thus it is often called “moving mean”. This moving mean has a very small frequency such that it does not excite neither the tower nor the conductors dynamically. However, the conductors are the element of the system, which can be excited by the turbulence associated with downbursts since their frequencies can be within the range of turbulences frequencies. Darwish et al. (2010) extracted the turbulence from the field measurements of a downburst conducted by Holmes et al. (2008). They used this turbulence measurement to study the dynamic response of the conductors. The study concluded that the dynamic response can be neglected due to the large aerodynamic damping of the conductors. In the numerical model developed by Shehata et al. (2005), the conductors were modeled using non-linear finite elements. A two-dimensional consistent curved beam element that was developed by Koziey and Mirza (1994), and then extended to include the geometric non-linear effect by Gerges and El-Damatty (2002), was used to model the conductors. Downbursts produce loading in both the transverse (horizontal) and vertical directions of the conductors. The analysis was decoupled between the two directions, and this could be justified since the dominant component is horizontal. By comparison, tornado associated velocities have three components of comparable magnitudes. As such, Hamada and El Damatty (2011) relied on FEA using a three-dimensional four-noded nonlinear cable element to model the conductors. The use of FEA for analyzing conductors under HIW imposes a challenge because of the extensive computational time. The analysis has to be carried in a nonlinear manner because of the high level of the conductor’s non-linearity. For downbursts, the transit nature of the loading requires a time history analysis. Moreover, because of the localized nature of the event and the extended length of the structure, the analyses have to
be repeated by changing the location of the event. Each location will correspond to a
different set of loading on the towers and conductors and the most critical location have
to be determined. Therefore, Aboshosha and El Damatty (2014) developed an effective
numerical technique to analyze multi-spanned conductors under varying loads along the
spans. Such variable loads are generated by High Intensity Wind (HIW) events. This
technique represents the first semi-closed form solution for a multi-spanned conductor
system under non-uniform loading in both the vertical and horizontal directions. The
solution accounts for the flexibility of the insulators, pretension forces and sagging.
Aboshosha and El Damatty (2015) derived a closed form solution for the reactions of
transmission line conductors. The solution allows evaluating the reactions under
downburst loads. The derivation was conducted using a simplified multi-spanned
conductor–insulator system. The solution was derived for downburst case with arbitrary
size and location. The proposed solution represents a practical and useful tool for
practitioner engineers.

Lastly, El Awady et al. (2017) conducted a novel experiment in this research program at
the WindEEE dome facility. The experiment involved testing a 1:50 multi-span aero-
elastic model under a simulated downburst. The experiments served to validate the
numerically predicted wind field and to estimate the turbulence characteristics of
downbursts. They assessed the dynamic response of a multi-span transmission line. A
decomposition approach was developed to separate between the resonant and the
background components of the response. The results were presented in the form of a
dynamic magnification factor that relates the peak response including the dynamic effect
to the maximum quasi-static response. El Awady et al. (2018) validated the numerical
model developed for the conductors (Aboshosha and El Damatty (2014)) and for the
conductor-tower system (Shehata et al. (2005)) using the aero-elastic model test results.

In addition to the author’s research group, other researchers conducted the following
studies related to the response of transmission lines to downbursts. The response of a
tower under microburst and tornado events was first evaluated by Savory et al. (2001).
Conductor forces were neglected and, as a result, failures were only associated with
tornado loading and no failure was observed with downburst loading. This is because
downbursts are larger in size and are expected to load a larger portion of the conductors compared to tornadoes. Wang et al. (2009) studied the dynamic effect of a downburst on transmission towers. The analysis showed that the dynamic effects were minor since the natural period of the tower is much lower than the natural period of the downburst loading. Mara and Hong (2013) studied the inelastic response of a transmission tower subjected to both a downburst and a synoptic wind field. The study showed a dependency of the tower capacity on the wind direction for both wind fields. Mara et al. (2016) assessed the load-deformation curve of a transmission tower under downburst wind loading, and compared it with that obtained for a normal wind loading profile. The analysis considers nonlinear inelastic response under simulated downburst wind fields. Yang and Hong (2016) reported the capacity curve of a single tower within the tower-line system under the downbursts considering both the mean and the turbulent wind components along with the tower-conductor interaction. The study employed the incremental dynamic analysis and the nonlinear static pushover analysis to estimate the capacity curve. Moreover, they conducted a comparison of capacity curve of a tower within a tower-line system to that of a single tower to determine the effect of the dynamic interaction between the tower and the conductors on the capacity.

It should be mentioned that up-to-current no loading provisions are available in the international code and guidelines to account for downburst, despite of the continuous failures observed worldwide for transmission lines under such events. The efforts conducted by the research group at the University of Western Ontario (UWO) led recently to the development of such downburst loading provisions for transmission lines reported by El Awady and El Damatty (2016) and El Damatty and El Awady (2018), which will be implemented in the American Society of Civil Engineering (ASCE-74 (2010)) guidelines of transmission lines loadings. Applying those loading provisions might allow designing the towers to resist downbursts. However, it might be expensive and not practical to design all the towers of a line for the large forces resulting from downbursts. Depending on the importance of the line and the available redundancy, an electrical company might accept the failure of one or multiple towers as long as failure is contained. To the best of the author’s knowledge, no numerical tool or commercial
software is available for conducting the analysis and predicting the failure that might progress from tower to another with a line segment under downbursts.

1.4 Objectives of Thesis

The main objectives of the thesis are:

1. Develop and validate a comprehensive numerical model capable of predicting the behavior and the progression of failure from tower to tower along a segment of a transmission line.
2. Apply the developed numerical model to a real transmission line system to gain an insight about its progressive failure and to assess the effect of various parameters on that failure.

1.5 Scope of Thesis

The main chapters of this thesis are integrated together in order to achieve the objectives of the thesis. In Chapter 2, a numerical model is developed to study the failure of a single tower. An important analytical development is then conducted in Chapter 3 through which the behavior of the conductors with non-aligned support condition can be predicted. This is important since post failure of a tower, the support of the conductors become non-aligned. Chapter four reports the details of the comprehensive numerical model, which incorporates the developments conducted in Chapter 2 and 3, together with further developments to carry on the progressive failure of a line consisting of multiple towers. A parametric study is conducted in Chapter 5 using the model developed in Chapter 4. Chapter 6 presents the conclusions from this study along with the suggestions for further research work.

1.5.1 Progressive Failure of Transmission Towers under Downbursts

In this chapter, a nonlinear finite element model is developed and validated to study the response of single towers and to assess their capacities under downbursts. This model accounts for geometric and material nonlinearity. The model is used to analyze a number of real self-supported and guyed transmission towers under downbursts. The critical downburst configurations are reported, the failure modes of the towers and the location at
which a mechanism is formed are investigated. The tower-failed members are strengthened by increasing their cross section in order to sustain the downburst forces. The increase in the weight of the structure associated with this strengthening is calculated.

1.5.2 Extensible Catenary Approach in Analyzing Transmission Line’s Conductors under Downbursts

In this chapter, a mathematical approach is developed to predict the structural performance of a cable with end points unaligned both vertically and horizontally. The approach takes into account the extensibility of the cable. Moreover, it considers the analysis of a multiple span cable, the flexibility of the cable’s supports and the effect of both in-plane and out-plane loads. An important application of this model is related to the performance of transmission line conductors under High Intensity Wind (HIW) such as downbursts. The localized nature of those events can lead to failure of one tower which results in misalignment of the end points of the adjacent spans. The developed model can be used in predicting the performance of an entire line post the failure of one tower during a downburst event. The validation of this mathematical model is conducted by comparing its numerical predictions to the results of the analysis of multi-span conductors using non-linear Finite Element Analysis (FEA). The main advantage of the mathematical model compared to FEA is the efficient computational time, which is very important for predicting the progressive failure of transmission lines under downbursts as this requires conducting a large number of analyses by varying the location and size of the localized wind event.

1.5.3 Numerical Model of Cascade Failure of Self-Supported Transmission Lines under Downbursts

In this Chapter, the numerical model developed in Chapter 2 and 3 are integrated together and extended to form a comprehensive numerical model capable of conducting progressive failure of a line consisting of multiple towers. The model involves conducting a parametric study to determine the critical downburst configuration for a specific tower in the line. Progressive failure analysis is then conducted to the tower under critical downburst configuration, and the location at which instability occurs is
determined. The movement of the unstable part of the tower increase the conductor forces, which can bring the tower again to a new state of equilibrium. The numerical model is capable of predicting such a behavior together with the conductor forces which are transferred to the adjacent towers. The numerical model then progress to study the behavior of adjacent towers and then the subsequent towers in the line. As such, this unique model can give a prediction and a snapshot for the states of the entire line segment under the effect of downbursts. A case study is considered to illustrate the application of this model, the behavior and failure models of the considered line are described.

1.5.4 Parametric Study on the Cascade Failure of Self-Supported Transmission lines under Downbursts

In this chapter, an extensive parametric study is conducted to identify the critical downburst configurations by varying the location of the downburst relative to the studied tower. In view of the critical configurations identified, a cascade failure analysis is conducted for a transmission line system consisting of eight spans and seven towers designed and constructed in accordance with current code provisions to withstand normal synoptic wind loading. The results of the parametric study, together with the progressive failure analysis, are reported for each tower. The results include the critical configuration of downbursts with respect to the structure, associated failure load, and a description of the failure mechanism. A parametric study is then conducted by varying the downburst jet velocities, span of the line and the insulator string lengths to study their effects on the cascade failure mechanism.

1.6 References


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Chapter 2

2 Progressive Failure of Transmission Towers under Downbursts

2.1 Introduction

Overhead transmission lines systems play an important role in operating a reliable electrical power distribution system. Extreme wind conditions are among the most severe environmental threat to the safe operation of the exposed tower-line systems. A large number of failures has been reported in the past due to weather conditions (Dempsey and White (1996), McCarthy and Melsness (1996) and Li (2000)). These include High Intensity Wind (HIW) events in the form of downbursts and tornadoes. In China, due to strong wind events such as downbursts, tornadoes and typhoons, Zhang (2006) reported failures of 18 towers belonging to 500 kV lines and 57 towers belonging to 110 kV lines. Most recently, 23 transmission towers collapsed during a series of downburst events in Southern Australia in September 2016 (Australian Wind Alliance, 2016). Also, several failures of transmission line system under downburst events have been reported in Canada such as the failure of 19 transmission towers located near Winnipeg, Manitoba reported by McCarthy and Melsness (1996), and the failure of two guyed towers belonging to Hydro One, Ontario (Failure report, 2006). Fujita (1990) defined a downburst as a sudden downfall of slow rotating air towards the ground. While reaching the ground, this sudden downfall bursts out violently causing an immediate rise in the wind velocity in the lower region of the ground. Fujita (1990) postulated that a downburst can produce wind gusts as high as 60-75 m/s and it can last for about 2-5 minutes. Moreover, downbursts have unique characteristics compared to synoptic winds such as hurricanes and typhoons. One of those characteristics is the localized nature of downbursts with respect to space and time. Transmission lines extending for thousands of kilometers are prone to failure under High Intensity Winds (HIW) by virtue of their length. The localized nature of the downbursts might result in a non-uniform and unsymmetrical distribution of the wind loads over the line spans. This results in load cases that do not usually exist under uniform and symmetrical large-scale wind events.
Lessons must be learnt from these catastrophic failures and it is important to understand the mechanism of loads acting on tower-line systems under these events and to investigate the causes of transmission tower collapses with a view to improve the design of transmission line structures. Few studies in the literature focused on assessing the behaviour of transmission tower systems under downbursts. The response of a latticed transmission tower under microburst and tornado events was first evaluated by Savory et al. (2001). In this study, the behaviour of the tower was investigated under specific downburst parameters. The downburst loading acting on the conductors was not considered in this study. Also, only the radial component of the microburst wind speed was considered. Shehata et al. (2005) developed a finite element numerical model that simulates transmission line system including the towers, conductors and insulators under downburst loading. One of the challenges of downburst related structural problems is that the acting forces vary with the characteristics of the downburst, such as its diameter and its jet velocity as well as the relative distance between the event and the structure. Therefore, Shehata and El Damatty (2007) conducted a parametric study by varying the location of the downburst center relative to the tower and varying the jet diameter ($D_J$) to determine the downburst critical configurations relative to the transmission tower. Afterwards, Shehata and El Damatty (2008) conducted progressive failure analysis of a guyed tower under the loads associated with the downburst critical configurations. Darwish and El Damatty (2011) investigated the behaviour of self-supported transmission towers under downbursts. A parametric study was conducted to determine the critical downburst configurations causing maximum axial forces for various members of a tower. The previous studies were conducted in a quasi-static manner given that the resonant component was negligible. One of the major differences between a synoptic and a downburst event is the non-stationarity nature of the latter. Downburst’s mean component varies with time, thus it is often called “moving mean or mobile mean”. This moving mean has a very small frequency such that it does not excite neither the tower nor the conductors dynamically. Nevertheless, the conductors are the element of the system prone to dynamic excitation by the turbulence associated with downbursts since their frequencies can be within the range of turbulence frequencies.
Wang et al. (2009) studied the dynamic effect of a downburst on transmission towers. The analysis showed that the dynamic effects were minor since the natural period of the tower is much lower than the natural period of the downburst loading. Darwish et al. (2010) extracted the turbulence from the field measurements of a downburst conducted by Holmes et al. (2008). They used this turbulence measurement to study the dynamic response of the conductors. The study found that there is almost no variation in the dynamic characteristics of the conductors under the different loading configurations. In addition, the study reported that the resonant component is negligible due to the large aerodynamic damping of the conductors. Lin et al. (2012) developed an aero-elastic model for a single span of a transmission line. The guyed lattice tower was simplified to an equivalent mast at a length scale of 1:100 while synoptic and downburst wind loading were applied with a time scale of 1:10. In either case of atmospheric boundary layer or downburst wind loading, the structural response was generally quasi-static. Resonant dynamic response was less evident with the downburst wind than with the synoptic wind. In a recent study conducted by Ibrahim et al. (2017), a dynamic analysis was conducted to predict the dynamic response of the conductor systems due to downburst loading and to examine the validity of using quasi-static analysis. The study revealed that the conductor system are dynamically insensitive and can therefore be treated quasi-statically.

Mara and Hong, (2013) studied the effect of wind direction with respect to the power line for a self-supported transmission tower and found that the critical wind speed initiating yield occurs for the transverse direction (wind perpendicular to the power line). Mara et al. (2016) assessed the load-deformation curve of a transmission tower under downburst wind loading, and compared it with that obtained for a normal wind loading profile. Their study showed that normal wind capacity curves can be used as an approximate alternative for those capacity curves resulting from downbursts. Yang and Hong (2016) reported the capacity curve of a single tower within the tower-line system under the downbursts considering both the mean and the turbulent wind components and the tower-wire interaction. The study employed the incremental dynamic analysis and the nonlinear static pushover analysis to estimate the capacity curve. Moreover, they conducted a comparison of capacity curve of a tower within a tower-line system to that of a single
tower to determine the effect of the dynamic interaction between the tower and the wires on the capacity. El Awady et al. (2017) conducted the first aero-elastic test under a scaled downburst wind field at the Wind Engineering, Energy and Environmental (WindEEE) dome facility. They assessed the dynamic response of a multi-span transmission line. A decomposition approach was developed to separate between the resonant and the background components of the response. The results were presented in the form of a dynamic magnification factor that relates the peak response including the dynamic effect to the maximum quasi-static response. Lastly, El Damatty and El Awady (2018) conducted one of the first studies that developed design load cases that will be incorporated in the American Society of Civil Engineering (ASCE-74). In addition, they assessed the economic impact of applying those load cases. However, it might be costly and not practical to design all the towers of a line for the large forces resulting from downbursts. Depending on the importance of the line and the available redundancy, the failure of one or multiple towers might be accepted as long as failure is contained.

Therefore, the current study focuses on developing a progressive failure model that can predict the tower’s capacities and failure mechanisms under downburst loading. Four transmission line systems are considered in this chapter as case studies. Using the developed numerical model, failure studies are conducted for each system. The failure studies included the critical downbursts configurations, selected in view of the parametric studies conducted.

2.2 Downburst wind field

The downburst wind field utilized in this study is based on Kim and Hangan (2007) CFD simulation. The resulting downburst velocity has two components: a radial (horizontal) component and an axial (vertical) component. The CFD model produced velocity field of these two components that vary in time and space depending on the jet diameter $D_J$ and jet velocity $V_J$. The current study adopted the scaling procedure proposed by Shehata et al. (2005) for the CFD data in order to estimate the spatial and time variations of the wind velocities associated with full-scale downbursts. The analysis assumed that the background component of the response was taken into account by scaling up the mean component to the gust wind speed. This means that the resonant contribution to the
The overall response of transmission line system subjected to downburst winds is assumed to be small and therefore neglected. This assumption coincides with the findings of previous research conducted by Lin et al. (2012), Aboshosha et al. (2015), Darwish et al. (2010) and EL Awady et al. (2017).

Figure 2-1 illustrates the profile of the radial velocity, normalized with respect to the jet velocity, along the height. The maximum radial velocity profile occurs at an R/D value of 1.2. On the other hand, Figure 2-2 shows the profile of the vertical velocity, normalized with respect to the jet velocity, along the height. The vertical component is quite small compared to the radial component at the first 100 m above the ground, i.e. within the height of typical transmission towers (see Figures 2-1 and 2-2). Figure 2-3 shows the radial velocity distribution along the full-scale time history of the downburst event. The figure shows that the radial velocity increases gradually until reaching a maximum peak and then decreases suddenly until reaching the minimum value, at which it remains constant throughout the rest of the time history. It is obvious from the previous discussion that the downburst velocities are influenced by the downburst location, which is determined by the variables (Dj, R/Dj and θ) and time history. Accordingly, an extensive parametric study should be considered in order to determine the most critical downburst configurations on the tower response.

Figure 2-1 Radial velocity profile along the height, Shehata et al. (2005)
2.3 Development and Verification of the Analytical Model

2.3.1 Numerical Model Development

Shehata et al. (2005) developed the basic finite element model that is used to predict the structural performance of a transmission towers as part of a transmission line system under downburst loading. A two-node linear three-dimensional frame element with three translational and three rotational degrees of freedom per node is used to model the tower members. Each tower member is simulated by one element. Rigid connections are
assumed between the tower members, as these are physically connected using multi-bolted connection that can transfer moments.

In the numerical model developed by Shehata et al. (2005), the conductors were modeled using non-linear finite elements. The conductor analysis should take into account their significant geometric nonlinear behavior and it should include the flexibility of insulators as well as the sagging and pretension forces. The analysis is conducted in a time history quasi-static manner. This makes the computational time quite significant. As mentioned earlier, the prediction of the peak responses of a tower to downbursts requires conducting an extensive parametric study by considering a large number of downburst configurations. All that makes the use of nonlinear finite element for the conductors modeling not practical. To solve this issue, Aboshosha and El Damatty (2014) developed an iterative analytical solution to obtain the response of multi-span conductors supported by insulators under both transverse and vertical non-uniform loading arising from downbursts. Shehata and El Damatty (2008) extended the capabilities of their numerical model to be able to conduct failure analysis of a single transmission tower under downbursts. Their model included the nonlinear material effect. Hamada and El Damatty (2015) extended this model to include the geometric non-linear effect. In the current study, both material and geometric nonlinear effects are included to study the failure of a tower under downbursts. In addition, different failure models were accounted for in this model including the (i) members axial capacity governed by the net section capacity in tension and buckling in compression (ii) connection rupture capacity including bearing and shear capacity.

2.3.2 Numerical Model Validation

Shehata et al. (2005) validated their model numerically and further validation was later conducted experimentally through the aero-elastic test conducted on a guyed tower using a unique experiment conducted by Elawady et al. (2017) at WindEEE dome testing facility at The University of Western Ontario in Canada, see Figure 2-4. Different downburst configurations were considered in the test and in the validation process. This validation was reported by Elawady et. al (2018) by comparing the aerodynamic forces, and straining actions predicted by the model to those measured during the test. A
comparison was first conducted between the base shears and base moments obtained from the experiment and the numerical models. The comparison considered both the mean and the peak responses of the tower. This provided a confidence in evaluating the aerodynamic forces under such a transient event. The second level of validation involved comparison of strains at different locations as well as the guys forces. This provided confidence in the accuracy of the finite element model. Regarding the conductor’s model, Aboshosha and El Damatty (2014) validated their analytical solution through comparison with non-linear finite element analysis. Further validation for this analytical solution was done through the aero-elastic tests conducted for a multi-span transmission line system under downbursts by Elawady et. al. (2017) at the WindEEE facility. The tests confirmed the accuracy of the analytical model in predicting the tension in the conductors under various downburst configurations.

![Figure 2-4 Aero-elastic TI testing at WindEEE dome, El Awady et al. (2017)](image)

Regarding the progressive failure model validation, a pushover analysis was conducted in the longitudinal and transverse directions on one of the existing towers owned by Hydro one, Ontario, Canada. The tower is a self-supported tower and has a total height of 54.65 meters and three cross arms as shown in Figure 2-5. In the validation process, a nonlinear Finite Element Analysis (FEA) was employed for the push over analysis. The
commercial software ABAQUS (2017) was utilized to perform the study. The material nonlinearity of the tower members is considered using no post yield model assuming that the fracture occur in the connections. The tower members are modeled using 2-node nonlinear 3-dimensional frame elements assuming rigid connections (representing multi-bolted moment-resisting connections). The tower model consists of 973 elements and 404 nodes. The geometric nonlinearity is accounted for through the use of a large deformation analysis. The validation is conducted by comparing the tower’s capacity curve obtained from the built in house numerical model to that obtained from ABAQUS. An excellent agreement between the two curves in the longitudinal and transverse directions can be shown in Figure 2-6(a-b), respectively. Further, the failed members are captured in the numerical model together with the ABAQUS model.

Figure 2-5 Isometric view of the tower model
Figure 2-6 Comparison between the pushover analyses conducted in both the numerical model and ABAQUS in (a) transverse direction (b) longitudinal direction
2.4 Description of the Considered Transmission line Systems

In the current study, four transmission line systems are investigated to assess the capacity and failure mechanisms under the downburst wind field: two guyed (G1&G2) and two self-supported (S1 &S2) towers. The studied transmission line systems consider the variations in: (i) the structural systems (guyed and self-supported), (ii) the tower and cross arm geometric configurations, (iii) conductor’s span and properties. The isometric view of the towers of the four line systems analyzed and reported in this study are shown in Figure 2–7.

The height of each tower, span of the wires as well as other conductors and ground wires’ properties are summarized in Table 2-1. The two guyed towers, G1, and G2 have different shapes. Tower G1 is slender and carries two conductors, while tower G2 is V shape, and carries three conductors. The self-supported towers S1 and S2 are similar in shape, and they each carry six conductors. However, a large difference exists between the wire spans of the two towers (213.6m for tower S1 and 450m for tower S2). It is assumed that the four tower systems considered in the study cover a wide variety of tower support systems, tower shapes, cross arm configurations, number of conductors and conductors' spans. They provide an accurate representation for high voltage lattice towers used in the industry.
Figure 2-7 Transmission line systems used in the failure study.
Table 2-1 Properties of selected towers

<table>
<thead>
<tr>
<th>Tower Type</th>
<th>G1</th>
<th>G2</th>
<th>S1</th>
<th>S2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tower Weight (KN)</td>
<td>34</td>
<td>78.2</td>
<td>96.8</td>
<td>78.1</td>
</tr>
<tr>
<td>Span (m)</td>
<td>480</td>
<td>460</td>
<td>213.36</td>
<td>450</td>
</tr>
<tr>
<td>Guy Diameter (m)</td>
<td>0.0165</td>
<td>0.0195</td>
<td>N.A</td>
<td>N.A</td>
</tr>
<tr>
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<td>46.57</td>
<td>54.65</td>
<td>51.81</td>
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<td>3</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>No of GWS</td>
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<td>2</td>
<td>2</td>
<td>2</td>
</tr>
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<td>Conductor Weight (N/m)</td>
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<td>8.67</td>
<td>28.97</td>
<td>20.14</td>
</tr>
<tr>
<td>GW Weight (N/m)</td>
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<td>5.45</td>
<td>10.4</td>
<td>3.823</td>
</tr>
<tr>
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<td>14</td>
<td>3.9</td>
<td>19.5</td>
</tr>
<tr>
<td>GW Sag (m)</td>
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<td>16</td>
<td>3</td>
<td>14</td>
</tr>
<tr>
<td>Design Velocity (m/sec)</td>
<td>32</td>
<td>36</td>
<td>45</td>
<td>34</td>
</tr>
</tbody>
</table>

2.5 Methodology

The current section reports the progressive failure analysis of the four considered towers under critical downburst cases. An extensive parametric study is conducted to determine the downburst critical configurations (R, D_j and \( \theta \)) that are likely to initiate failure of a number of critical members of the tower as shown in Figure 2-8.

The most critical downburst configuration is considered in the failure study of each tower. Then a nonlinear analysis is conducted for each critical downburst configuration. The tower is being solved incrementally and once the member capacity is reached in one of the increments, the member is eliminated from the structural stiffens matrix in the subsequent increments. The deformed shape is captured incrementally and an intermediate hinge formation can be observed when the structure has lost its overall stability.

For each critical configuration, the following steps are conducted in the progressive failure analyses:

1. A parametric study was conducted using the finite element model to determine the maximum axial forces in each member and its critical downburst configuration (\( \theta \), R / D_j and D_i). The parametric study was conducted using different values for the jet
velocity \((V_1 = 30, 40 \& 50 \text{ m/s})\). For a specific downburst configuration, the internal forces on the selected tower members are evaluated instantaneously throughout the entire time history of the downburst event. The analysis is repeated with different downburst configurations, the maximum forces on each member are then evaluated.

2. For each critical downburst configuration, a nonlinear analysis is conducted during each increment along the downburst time history. The internal forces are calculated for all members of the tower.

3. In the time history analysis, a member reaching its ultimate capacity at a certain increment means it is eliminated from the stiffness matrix in the subsequent increments. The capacity of the member is determined by the least member’s axial capacity and the connection rupture capacity; which is measured by the least of the connection bearing capacity and the connection shear capacity.

4. The structure loses its overall stability and a state of collapse is assumed when no convergence in the numerical solution is reached.

![Figure 2-8 Downburst characteristics parameters](image)

2.5.1 Failure Modes and Intermediate hinges

In order to explain the failure modes for the different towers, a description of the structural system is first provided in Figure 2-9. The guyed towers (G1&G2) can be
treated as an overhanging beam, where the cantilever portion is located above the guys-tower attachment point. The towers have a pin support at the base and a flexible support at the supporting guy’s cross arms location. A major difference between towers G1 and G2 is that for tower G2, the conductors and supporting guys are both attached to the same cross-arms. The self-supported towers (S1&S2) are presented as a cantilever beam fixed at the base. The downburst loading acting on the structural system is represented as a distributed load along the tower height and concentrated forces transferred from the conductors and the ground wires to the tower through the insulators.

![Image](image-url)

**Figure 2-9 Structural System (a) guyed towers (b) self-supported towers**

### 2.5.1.1 Failure Analysis of G1 Transmission Tower

The failure of the guyed tower G1 was associated with the transverse downburst configuration \(D_J = 750, R/ D_J = 1.2, \theta = 0^\circ, 180^\circ\). The location of the downburst relative to the tower is shown in Figure 2-10. In this case, the conductors on either sides of the tower will be subjected to large and equal downburst transverse loads leading to a large
resultant force acting on the cantilever top part of the tower in a direction perpendicular to the conductors. This force will cause a large negative bending moment in the cantilever part of the tower as well as a large shear force in the guys’ area. As shown in Figure 2-11, failure initiates at the diagonal members at the guys’ cross-arm region (stages I). Large external shear forces in this area are responsible for the failure of this diagonal member. Failure of the main chord members occurs at the subsequent load increment (stages II) as shown in Figure 2-11. The supporting guys on the leeward side start slacking. This results in a change in the supporting system of the structure and a redistribution of the internal forces. Other chord members start to fail and an overall collapse then occurs (stages III).

Figure 2-10 Plan view for the critical downburst location.
2.5.1.2 Failure Analysis of G2 Transmission Tower

For tower G2, where one cross arm is used for both the guys and the conductors, the analyses show that the conductor’s forces have a negligible effect on the tower members since a large percentage of the conductor’s forces transfers directly to the ground supports through the guys. As such, the conductor’s forces have a minimal effect on the internal forces developing in the tower members. However, for this tower, the aerodynamics of the tower face, which is perpendicular to the line direction, are larger than those of the tower face which is parallel to the line direction, width of the conductors cross arm is 29.34 m. Therefore, most of the shaft members experience their peak internal forces at an angle of attack of $\theta =90^\circ$. The failure of the guyed tower G2 was associated with the longitudinal downburst configuration ($D_1=750$, $R/ D_1 =1.2$, $\theta =90^\circ$, $270^\circ$). The location of the downburst relative to the tower is shown in Figure 2-12.
Due to the downburst longitudinal forces, supporting guys 1 & 2 are expected to slack while guys 3&4 are expected to be subjected to a large tensile force as shown in Figure 2-13. It can be observed in Figure 2-13 that there are two intermediate hinges formed in the upper two thirds of the two truss columns just below the cross arm, whereas G2’s truss columns has approximately the same structural stiffens along the height. G2 can be presented as a simply supported beam with a relatively small overhanging beam. Due to the non-uniform downburst loading which keeps increasing from the base till the tip of the tower.it is expected that the maximum moment will occur at the upper two thirds of it.
2.5.1.3 Failure Analysis of S1 Transmission Tower

Self-supported tower behaves like a cantilever. As such, the peak straining actions in the tower occur when both the tower and the conductors are fully loaded by downburst wind forces, i.e., at $\theta = 0^\circ$. Therefore, the failure of S1 was associated with a transverse downburst configuration ($D_j = 750$, $R/D_j = 1.2$, $\theta = 0^\circ$, $180^\circ$), the location of the downburst relative to the tower is shown in Figure 2-10. This load configuration leads to a maximum transverse loads along the height of the tower as well as the conductor attached to the tower. This is consistent with the findings in the literature, Mara and Hong (2013) studied the effect of wind direction with respect to the power line for a self-supported transmission tower and found that the critical wind speed initiating yield occurs for the transverse direction (wind perpendicular to the power line). S1 is expected to fail near the base where the maximum bending moments are located. This is illustrated in Figure 2-14, where the two diagonal members highlighted failed at stage (I) and then two main chords failed at stage (II). Afterwards, the failure propagated in stage (III) along the tower height due to buckling of the chord members on the leeward side. An
intermediate hinge is formed in the lower main chords just above the base where the maximum bending moment is located because of the cantilever action.

![Figure 2-14 S1 Progressive failure and intermediate hinge Formation](image)

**Figure 2-14 S1 Progressive failure and intermediate hinge Formation**

### 2.5.1.4 Failure Analysis of S2 Transmission Tower

Similarly, the self-supported tower S2 failed due to the same downburst critical configuration ($D_j = 750$, $R/D_j = 1.2$, $\theta = 0.180$), where the maximum transverse loads along the height of the tower as well as the conductor attached to the tower are obtained. The peak internal forces in the chord members of the self-supported tower S2 tends to be critical just below the third cross arm. This is expected since the shear and moment keeps increasing from the tip of the tower and there is an abrupt increase at each cross arm level due to the conductor forces. Accordingly, shear and moment are being maximized just below the third cross arm. As shown in Figure 2-15, S2 failed due to intermediate hinge
formation on the leeward side just below the third cross arm, waist level, where the chord members experienced maximum axial compression stresses at stage (II).

![Figure 2-15 S2 Progressive failure and intermediate hinge formation.](image)

### 2.6 Tower Strengthening

As mentioned in the previous section, the failure analysis is used to evaluate the maximum internal forces resulting from downbursts in the members of the towers of the four considered systems. These internal forces are compared to the members’ capacity. The ratio between the acting force and the strength is evaluated for all members. When this ratio is found to exceed a unity, the cross-section of the member is upgraded such that this ratio becomes slightly above unity. The sequence of the tower strengthening is shown in Figure 2-16. The analysis begins with a certain $V_J$ (30, 40 or 50) m/sec. First, a parametric study is conducted to measure the downburst critical configurations ($D_J$, $R/$
Dₙ, θ). Second, a nonlinear analysis is conducted to assess the transmission towers capacity under downburst loading. If the tower exhibits local or global failure mechanisms, the failed members are strengthened and upgraded. After the tower is strengthened, the analysis is repeated, starting from the parametric study that may result in different downburst critical configurations, followed by failure analysis, and strengthening if failure happened. The analysis is repeated until the tower is capable of resisting all the downburst critical cases. Last, the weight of the upgraded tower is evaluated and compared to the initial weight.

As shown in Figure 2-17 I, the self-supported tower S1 first intermediate hinge formation was in the lower main chords just above the base. The tower is first strengthened by updating the failed member’s capacity, and then the parametric study and the failure analysis are repeated. It is obvious that an intermediate hinge formation is transferred

**Figure 2-16 Flow chart summarizing the analysis steps conducted for the tower strengthening**

As shown in Figure 2-17 I, the self-supported tower S1 first intermediate hinge formation was in the lower main chords just above the base. The tower is first strengthened by updating the failed member’s capacity, and then the parametric study and the failure analysis are repeated. It is obvious that an intermediate hinge formation is transferred
from the lower zone to the tower’s main shaft between the second and the third cross arm as shown in Figure 2-17 II. The members are then updated, the analysis is conducted again and an intermediate hinge is transferred to the tower’s main shaft just above the second cross arm and below the first cross arm as shown in Figure 2-17 III. If no failure components appear, all the added members are calculated and compared to the initial weight. Torsional failure mechanisms may occur if the failed members were not strengthened equally. As shown in Figure 2-18, one of the two truss columns in G1 was strengthened more than the other one; as a result, the tower was twisted when subjected to the downburst critical configuration (D1 =750, R/ D1 =1.2, θ =0º). Similarly, one of the G2’s main chord was strengthened more than the other main chords; therefore, two main chords exceeded their structural capacity and the main tower shaft was twisted.

**Figure 2-17 S1 strengthen and intermediate hinge formation.**
2.6.1 Strengthening For the Self-Supported Tower S1&S2:

As mentioned in the previous section, the peak internal forces in the chord members of the self-supported towers S1\&S2 tends to be critical right below the third cross arm. This is expected since the self-supported tower acts as a free cantilever with a maximum straining actions happening at the fixation zone. The analysis shows that in order for these chords to resist the downburst load with a jet velocity of (30, 40 and 50 m/sec), an increase in the weight of those members of approximately (2\%, 15\% and 42\%) is needed respectively for S1 tower strengthen. While, S2 requires (0\%, 4\% and 12\%) respectively to resist the same loading. Although the two towers are self-supported and are subjected to the same downburst loading, S1 requires more strengthening than S2. This was expected since S1’s mean design velocity (V_d) under normal wind is 34 m/sec. While,
S2’s design velocity is 45 m/sec. Since the two towers were designed using different $V_d$, a ratio between the downburst horizontal velocity ($V_h$) and ($V_d$) was used to provide common ground for comparison. Figure 2-19 illustrates the strengthening ratio as opposed to the increase in $V_h/V_d$. A value for $V_h/V_d$ that is equal or less than 1 means that the tower should be capable of withstanding the downburst loading and there is no need to add more members. Comparing for a $V_h/V_d$ that is equal to 1.3, S2 requires 11% strengthening while S1 requires 10.5%, which is relatively close.

![Graph](chart.png)

Figure 2-19 Percentage of weight added to the self-supported towers (S1&S2) vs $V_h/V_d$ ratio.
2.6.2 Strengthening For the Guyed Towers G1&G2

As mentioned previously, the peak internal forces in the diagonal and chord members of the guyed tower G1&G2 tends to be critical at the guy’s cross arm location. This is expected since the guyed tower acts as an overhanging beam, where the cantilever portion is located above the guys. As such, the maximum straining actions is expected to occur near the middle part of the beam. As illustrated in Figure 2-20, the analysis shows that in order for these members to withstand a downburst jet velocity of (30, 40 and 50 m/sec); an increase in the weight of those members of approximately (0%, 4.8% and 10%) is needed respectively for G1 tower strengthen. Meanwhile, G2 requires (0%, 10% and 25%) respectively to resist the same downburst jet velocities. Although the two towers (G1 &G2) have relatively the same design speed (32, 36 m/sec), respectively. The significant increase in the G2 tower’s weight can be attributed to the geometric configuration and the number of bundles carried by each tower. G2 has two separate truss columns connected with a wide cross arm 29.34 m, which is larger than G1’s cross arm 13.3m. This will result in more downburst critical configurations to overcome, especially in the longitudinal direction. Moreover, G2 carries one more ground wire and one more conductor. The spatial variation in the location of the parallel lines, resulting from the wide cross-arm, can lead to different values for unbalanced longitudinal forces acting on the cross-arms, which will cause a net torsion effect on the tower.
2.7 Conclusion

The present chapter demonstrates the progressive failure analysis and the strengthening of the guyed and self-supported transmission towers under downbursts. A nonlinear finite element model is developed to study the response of the towers and to assess their capacities under downbursts. The study shows that overhead transmission towers are not capable of resisting the downbursts loading. Self-supported tower (S1, S2) and guyed tower G1 failure mechanisms are due to the downburst critical configuration when it is perpendicular to the transmission line. Meanwhile, the guyed tower G2 failure mechanisms occur when the downburst is on the centerline of the transmission line. An intermediate hinge is formed just below the lowest cross arm in the self-supported towers on the leeward side, where maximum compression stresses occur in the diagonals and

Figure 2-20 Percentage of weight added to the guyed towers (G1&G2) vs Vh/Vd ratio.
inelastic buckling in the chord members. S1 exhibits local failure mechanism just below the lowest cross arm; S2 exhibits a global failure mechanism starting from the lowest cross arm and propagating to the base because of the cantilever action. In addition, the guyed tower G1 intermediate hinge is formed in the guy’s cross arm and is propagating in the tower’s shaft members. G2 intermediate hinge is formed in the upper two thirds of the two truss columns, below the cross arm.

The failed towers are strengthened, and the weight added to the four towers to sustain the critical downburst configuration is measured. A downburst jet velocity of 30, 40 and 50 m/sec is assumed. For the guyed tower systems, the increase in the weight is 0%, 5% and 10%, respectively, for G1, and 0%, 10% and 25%, respectively, for G2. The significant increase in the G2 tower’s weight can be attributed to the different geometric configuration and to the more conductor bundles carried by this tower. Meanwhile, the weight needed for the self-supported towers are 2%, 15% and 42%, respectively, for S1 and 0%, 4% and 12%, respectively, for S2. It can be concluded from the significant weight added to the self-supported towers- except S1, which is originally designed on a high wind speed 45 m/sec - when compared to the weight added to the guyed towers that conventional guyed tower design is more downburst resistant than the self-supported towers due to their flexibility and lightweight.

2.8 Acknowledgment

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2.9 References


Chapter 3

3 Extensible Catenary Approach in Analyzing Transmission Line’s Conductors under Downbursts

3.1 Introduction

This study is a part of an on-going extensive research program that started more than fifteen years ago at the University of Western Ontario, Canada, focusing on the impact of downbursts and tornados on transmission line (TL) structures. Downbursts and tornados form together a category of wind storm often labelled as High Intensity Wind (HIW) events. Fujita (1985, 1990) defined a downburst as a mass of cold and moist air that drops suddenly from the thunderstorm cloud base impinging on the ground surface and then transferring horizontally.

Dempsey and White (1996) reported that HIW are responsible for more than 80% of transmission line towers failures worldwide. McCarthy and Melsness (1996) reported the failure of 19 transmission line towers due to downburst events in Canada. Li (2000) reported that downbursts are responsible for more than 90% of weather-related failures of structures in Australia.

Conductor’s loading constitutes a significant portion of the loads acting on a transmission tower. The studies conducted by Shehata et al. (2007), Aboshosha and El Damatty (2012) and Elawady et al. (2017) emphasized the significant contribution of the conductor loads on the failure of transmission towers during downburst events.

Regarding the conductor analysis, Winkelman (1995) developed the ruling span concept, which is widely used to calculate sags and tensions for overhead transmission lines. It provides satisfactory results for a levelled line. The ruling span formula is based on the fundamental assumption that the attachments of the conductor to suspension structures are supported by an element that is infinitely flexible in the longitudinal direction. This is not applicable for the loading cases involving longitudinal loads, differential temperature (Motlis et al. (1998)) and HIW events (El Damatty and EL Awady (2018)). Moreover,
the ruling span concept will result in an unacceptable magnitude of error if it is used to calculate sags and tensions in a line segment with significantly unequal spans. Keshavarzian and Priebe (2000) presented a method to calculate sags and tensions of multi-span line segments at different temperatures based on the rotational stiffness of suspension insulator strings. They derived a simple equation to calculate changes in the span lengths and conductor sags and tensions. Because of the limitation of the ruling span method in accounting for longitudinal loads, Finite Element Analysis (FEA) was utilized to analyze TL’s conductors under downbursts. In the work done by Shehata et al (2005), a two-dimensional consistent curved beam element that was developed by Koziey and Mirza (1994), and then extended to include the geometric non-linear effect by Gerges and El-Damatty (2002), was used to model the conductors. Downbursts produce loading in both the transverse (horizontal) and vertical directions of the conductors. The analysis was decoupled between the two directions, and this could be justified since the dominant component is horizontal. By comparison, tornado associated velocities have three components of comparable magnitudes. As such, Hamada and El Damatty (2011) relied on FEA using a three-dimensional four-noded nonlinear cable element to model the conductors. The use of FEA for analyzing conductors under HIW imposes a challenge because of the extensive computational time. The analysis has to be carried in a nonlinear manner because of the high level of the conductor’s non-linearity. For downbursts, the transit nature of the loading requires a time history analysis. Moreover, because of the localized nature of the event and the extended length of the structure, the analyses have to be repeated by changing the location of the event. Each location will correspond to a different set of loading on the towers and conductors and the most critical location have to be determined. Therefore, Aboshosha and El Damatty (2014) developed an effective numerical technique to analyze multi-spanned conductors under varying loads along the spans. Such variable loads are generated by High Intensity Wind (HIW) events. This work presents the first semi-closed form solution for a multi-spanned conductor system under non-uniform loading in both the vertical and horizontal directions. The developed technique accounts for the flexibility of the insulators. However, it is limited in its application to levelled spans.
Other than FEA, various methods of approximation ((Enos (1949), Barthold et al. (1993), Paul Cella (1999)) have been proposed for estimating the geometric and mechanical elements of a catenary. “Catenary” is the term given to the curved shape of conductors hanging between the support points of towers. Other researchers (Irvine (1981) Yu et al. (1995)) derived a closed-form solution for the reactions of a single-spanned conductor without taking into account the flexibility of the insulators. As indicated by Darwish et al. (2010), insulator flexibility is an important parameter in quantifying the amount of forces transferred to the towers. Sakala (2016) improved the calculation of sag for a one span conductor supported at unequal heights by applying the Newton Raphson method. The review of the literature indicates that no analytical approach was developed to analyze multiple conductors with unlevelled ends, supported by elastic supports, and subjected to both in plane and transverse loads.

This study is motivated by a situation that can happen as a result of severe downbursts. Because of the localized effect of those events, failure can occur progressively in the members of one tower of a line. The failure will be triggered when the downburst is at a location critical to this particular tower. The progressive failure of the members of the tower will eventually lead to the formation of an intermediate hinge, which will result in the formation of a mechanism. All this stage, the unstable part of the tower will start to rotate. Consequently, the conductors attachment point to the tower will move vertically and horizontally leading to a change in the conductor’s tension force. The increase in the conductor tension will tend to stabilize the unstable part of the structure leading eventually to another equilibrium state. The evaluation of conductor’s forces is important, as it will be transferred to the adjacent towers, which will be subjected to unbalanced longitudinal forces that might lead to a cascade failure of those towers. As such, the objective of this chapter is to develop and validate an analytical procedure that can predict the conductor tension with end conditions that are not aligned both horizontally and vertically while being subjected to its own weight and transverse wind loads. The availability of such a procedure will make the prediction of the cascade failure of an entire line during downbursts possible. The chapter starts by presenting briefly the catenary solution of a single span conductor with levelled ends under its own weight. This classical well-established solution presents the basic of the analytical development
carried out in the chapter. The solution is extended to the case of vertical misalignment of the conductor under its own weight and then under the combined effect of own weight and transverse wind loads. Finally, the last case is extended to include both vertical and horizontal misalignments. The developed analytical solution is validated using a benchmark problem solved using FEA. The analytical solution is extended to the case of multi-span conductors supported by flexible insulators with unaligned supports horizontally and vertically and subjected to both own weight and transverse wind loading. A multi span conductor with such support conditions and subjected to downburst wind load is analyzed using the developed procedure and then using FEA for validation.

3.2 Formulation of a Single Span

3.2.1 Levelled Span under In-Plane Loading

When a conductor hangs between two horizontal supports and is subjected to in-plane loads only, it takes the form of a curve, which is called catenary. A span having equally levelled supports is called a “level span”, whereas when the level of the supports is not the same, the span is called “unlevelled span”. The analysis of a level span conductor under in-plane loads is provided below.
This is the basis of the formulation derived in this chapter. Consider a conductor AOD suspended freely between two levelled supports, A and D. The lowest point of the conductor is O as shown in Figure 3-1.

Knowing the length of the curve $S_L$, the horizontal distance between the supports $A_lx$, the weight per unit length, $w$, the objective is to obtain the tension, $T$, at any point in the cable (which act in the direction of the slope of the curve at that point) and the sag value $h_L$. The horizontal tension, $T_o$, can be then obtained.

The solution of this problem can be found in Higdon and Stiles (1995). In the coordinate system shown in Figure 3-1, the origin “O” is at the lowest point, and $S$ is a coordinate representing the curved length of the curve between point “O” and any point in the curve having coordinates $X$ and $Z$. The catenary solution provided by Higdon and Stiles (1995) leads to the following equations:

$$S = C \sinh(x/c)$$  \hspace{1cm} (3-1)

$$Z = C \left[ \cosh(x/c) - 1 \right]$$  \hspace{1cm} (3-2)

where $C = T_o/w$  \hspace{1cm} (3-3)
Substituting the coordinate of point “D” into Eq. (2-1) and Eq. (3-2), which are: \( x = \frac{A L x}{2}, Z = h_L \) and \( S = S_l/2 \), leads to the following equations:

\[
S_l = 2C \sinh(\frac{A L x}{2C})
\]  
(3-4)

\[
h_l = C [\cosh(\frac{A L x}{2C}) - 1]
\]  
(3-5)

Knowing \( S_L \), Eq. (3-4) can be used to obtain \( C \), which can be then substituted into Eq. (2-5) to obtain the sag value \( h_L \). Once \( C \) is known, the horizontal tension \( T_o \) can be obtained from Eq. (3-3). The resultant tension at any point can be then evaluated using the following equation given by Higdon and Stiles (1995)

\[
T = T_o \cosh(\frac{x}{c})
\]  
(3-6)

The above solution does not take into account the extension of the cable assuming that it is fully rigid. This can be taken into account by adopting an iterative procedure, while considering a variation in the length of the curve. \( S_L^{(i)} \) represents the length of the curve at iteration \( i \) while \( S_L^{(0)} \) represents the initial length of the curve. At iteration \( i \), the average tension along the conductor span \( T_{mn}^{(i)} \) can be calculated by first integrating the tension along the length of the span to obtain \( T_{total}^{(i)} \)

\[
T_{total}^{(i)} = \int_{-S_l/2}^{S_l/2} T^{(i)} \, ds
\]  
(3-7)

Substituting Eq. (2-6) into Eq. (2-7) and using Eq. (3-1) to change the integration from \( ds \) to \( dx \), leads to

\[
T_{total}^{(i)} = \frac{T_{l}^{(i)} S_l^{(i)}}{2} + \frac{T_o A L x}{2}
\]  
(3-8)

As such, the average tension \( T_{mn}^{(i)} \) is given by

\[
T_{mn}^{(i)} = \frac{1}{2} \left[ T_o A L x + T_{l}^{(i)} \frac{S_l^{(i)}}{S_l^{(i)}} \right]
\]  
(3-9)
Given the conductor’s cross-sectional area, \(A_c\), and its Young’s modulus, \(E\), the stresses \(\sigma(i)\) and the corresponding strains \(\varepsilon(i)\) can be calculated as follows:

\[
\sigma(i) = \frac{T_{mn}(i)}{A_c}
\]

(3-10)

\[
\varepsilon(i) = \frac{\sigma(i)}{E}
\]

(3-11)

The new length \(S_L^{(i+1)}\) can be then calculated as:

\[
S_L^{(i+1)} = S_L^{(i)} \left(1 + \varepsilon(i)\right)
\]

(3-12)

With the new conductor length \(S_L^{(i+1)}\) the procedure can be repeated to obtain the tension \(T^{(i+1)}\), the average tension \(T_{mn}^{(i+1)}\), the strain \(\varepsilon^{(i+1)}\) and the new conductor’s length \(S_L^{(i+2)}\). The procedure can be repeated until the difference in the length of the conductor between an iteration and the previous one become less than a certain tolerance limit.

### 3.2.2 Vertically Unlevelled Span Under In-plane Loading

The solution for levelled spans under in plane loading will be extended to consider the case of vertically unlevelled span under the same loading. Figure 3-2 shows the dimensions and variables associated with this case. This problem can be treated as two catenary problems, left and right of the lowest point “O”. In view of the levelled span solution, the set of equations for the left segment are:

\[
S_{l1} = C \sinh(X1/C)
\]

(3-13)

\[
h_{l1} = C \left[\cosh(X1/C) - 1\right]
\]

(3-14)

Four unknowns exist in Eq. (3-13) and Eq. (3-14), which are \(S_{l1}\), \(h_{l1}\), C and X1. Also, the levelled span solution gives the following set of equations for the right segment:

\[
S_{l2} = C \sinh(X2/C)
\]

(3-15)

\[
h_{l2} = C \left[\cosh(X2/C) - 1\right]
\]

(3-16)
Notice that “C” is the same for both the left and right segments since the horizontal tension “To” is constant along the entire span. Eqs. (3-13)-(3-16) have together seven unknowns $S$, $S_{L1}$, $h_{L1}$, $X1$ $S_{L2}$, $h_{L2}$ and $X2$. As such, three more equations are needed to solve this problem. Referring to Figure 2-2, those three extra equations result from geometric compatibility and they are:

\[
\Delta Z = h_{L2} - h_{L1} \tag{3-17}
\]

\[
S = S_{L1} + S_{L2} \tag{3-18}
\]

\[
ALx = X1 + X2 \tag{3-19}
\]

Where $ALx$ and $\Delta Z$ are known given the location of the end supports and $S_L$ is the entire length of the cable. The effect of the extension of the cable can be accounted for similar to what was done for the levelled case by conducting an iterative solution involving calculating the average tension, the equivalent strain and elongation at each iteration, and updating the total length of the cable until the solution converges to a certain value.

In view of the solution provided in the previous sub-section, it can be found that the average tension $T_m^{(i)}$ at iteration (i) is given by:

\[
T_m^{(i)} = \frac{1}{2} \left[ \frac{T_o ALx + T_{L1}^{(i)} S_{L2} + T_{L2}^{(i)} S_{L2}}{S} \right] \tag{3-20}
\]

The corresponding strain $\varepsilon^{(i)}$ and the deformed cable length $S_L^{(i+1)}$ are as given in Eq. (3-11) and Eq. (2-12) and the iterative procedure follows what is described for the levelled case.
3.2.3 Vertically Unlevelled Span under In-Plane and Transverse Loads

Consider the vertically unlevelled cases, subjected to both vertical load $W$, and transverse load $H$. $R$ is the resultant of the loads as shown in Figure 3-3. The angle “θ” is given by:

$$\theta = \sin^{-1} \left( \frac{W}{R} \right)$$ \hspace{1cm} (3-21)

The conductor will rotate about the pivot line AD as shown in Figure 3-3, ADC represent the initial plan of the conductor, while $\text{ADC}'$ represents the conductor’s plane after rotation. “θ” is the angle of inclination of $\text{ADC}'$ with the horizontal plane. The same solution procedure carried out in the previous section can be adopted in this case, while considering the geometry of the cable in the inclined plane $\text{ADC}'$. The distance $DC$
represents “\(\Delta Z\)”, which is the vertical misalignment between the end points of the cable. \(DC'\) is the projection of this distance in the inclined plane of the conductor after being subjected to transverse loading. In view of Figure 3-3, the relation between \(DC'\) and \(DC\) is:

\[
DC' = \Delta Z \sin \theta = DC \sin \theta = \Delta Z'
\]

(3-22)

The distance \(AC\) represent \(ALx\) used in the solution provided in the previous section. \(AC'\) is the corresponding value in the inclined plane, which can be obtained from the following relation

\[
AC' = \sqrt{AD^2 - DC'^2} = Alx'
\]

(3-23)

Knowing \(\Delta Z'\), and \(Alx'\), the procedure given in subsection 3.2.2 can be used to solve the problem to obtain the cable geometric and the tension at any point by replacing \(\Delta Z\) with \(\Delta Z'\) and \(Alx\) with \(Alx'\).

![Diagram](image)

**Figure 3-3** unlevelled conductor's profile in Z direction under vertical and lateral loads

### 3.2.4 Vertically and Horizontally Un-levelled Span under In-Plane and Transverse Loads

Figure 3-4 shows the case of both vertical and horizontal misalignment. One end of the cable is located at point “A” while the second end at point “D’”. The cable will rotate...
about the pivot line $AD'$ and it will be located in the plane $AD'C'$. The distances $\Delta Z$ and $\Delta Y$ shown in the figure represent the vertical and the horizontal misalignments, respectively. The solution can be carried out in $AD'C'$ plane with

$$\Delta Z' = D'C' = \Delta Z \sin \theta - \Delta Y \cos \theta$$  \hspace{1cm} (3-24)

Where $\theta$ is the angle of inclination of the plane at the conductor due to the application of both vertical Load “W” and transverse load “H”, as defined in the previous subsection. Also, from Figure 3-4,

$$Alx' = AC' = \sqrt{(AD'^{2} - D'C'^{2})}$$  \hspace{1cm} (3-25)

Again, the solution can be carried out as described in subsection (3.2.2) using $\Delta Z'$, and $Alx'$, calculated above, instead of $\Delta Z$ and $Alx$.

![Figure 3-4 unlevelled conductor's profile in Z & Y directions under vertical and lateral loads](image-url)
3.2.5 Validation of Single Span Solution

In order to validate the developed analytical solution, a single span conductor is analyzed using finite element modeling and the numerical solution is compared to that predicted by the analytical method. The considered conductor has a length of 305 m, weight of 20 N/m and is subjected to wind load at 15 N/m. The conductor is suspended between supports separated by 300 m horizontally, 36 m vertically and 18 m transversely. The conductor’s cross section area is 6.452E-04 m² and its Young’s modulus is 6.48E+10 N/m². The commercial software SAP 2000 is utilized to perform the FEA, using two nodded nonlinear cable elements to simulate the conductors. The conductor’s reactions (Rx, Ry and Rz) are shown in Table 3-1 along with the FEA results. As shown in the table, the reaction components at both ends of the conductor predicted by the analytical approach and FEA are almost identical. This provides a validation for the analytical method in solving single span unlevelled conductors subjected to in plane and transverse loading.

Table 3-1 One span nodal reactions

<table>
<thead>
<tr>
<th></th>
<th>SAP 2000 (N)</th>
<th>Extensible Catenary (N)</th>
<th>Error %</th>
</tr>
</thead>
<tbody>
<tr>
<td>RX1</td>
<td>-17040.0</td>
<td>-17045.3</td>
<td>0.03</td>
</tr>
<tr>
<td>RY1</td>
<td>3300.1</td>
<td>3301.3</td>
<td>0.03</td>
</tr>
<tr>
<td>RZ1</td>
<td>992.0</td>
<td>992.6</td>
<td>0.06</td>
</tr>
<tr>
<td>RX2</td>
<td>17040.1</td>
<td>17045.3</td>
<td>0.03</td>
</tr>
<tr>
<td>RY2</td>
<td>1274.9</td>
<td>1275.6</td>
<td>0.05</td>
</tr>
<tr>
<td>RZ2</td>
<td>5107.9</td>
<td>5109.8</td>
<td>0.03</td>
</tr>
</tbody>
</table>

3.3 Analytical Solution For Multi-Span Conductors Supported by Insulators

The lines of transmission structures often consists of multi-span conductors supported by insulators. As indicated by Darwish et al. (2010), insulator’s flexibility is an important parameter in quantifying the magnitude of conductor’s tension and consequently the forces (Rx, Ry and Rz) transmitted to the towers and, thus cannot be ignored. As such, the analytical solution is extended to consider the line shown in Figure 3-5. The figure shows a multi span conductor supported by insulators with vertical misalignments of the support points. The solution considers horizontal misalignment as well of the support points. The system has spans with length, ALx, and sag, hL. Each span is supported by
two insulators having each a length, L. The insulators are assumed to be axially rigid. The system is subjected to loads in the transverse direction Y defined as H and in the vertical Z direction defined as W. As a result, the conductor system will have displacements and reactions in the X, Y and Z directions.

Figure 3-5 Multiple Conductor spans with vertical and horizontal misalignments (a) a cut in the insulator conductor connecting point (b) Insulator equilibrium state

The analysis is performed by dividing the system into a number of single spans labeled in Figure 3-5 as (i-1), (i) and (i+1). Also, the figure shows labels for three conductor insulator connecting points labeled as (I-1), (I) and (I+1). The pinned ends of the insulators represent their connection points to the tower’s cross arms. Their locations are in generally unaligned vertically and horizontally. The other ends of the insulators represent the attachment points to the conductors. As shown in Figure 3-5, for a
conductor “i”, the two attachment points to the conductor are labelled as $A_i$ and $B_i$. The location of those points depends on the insulator cross arm points (assumed to be pinned) and the rotation of the insulators. The tension in the conductors defines the insulators rotation and consequently the location of points $A_{i-1}$, $A_i$, $A_{i+1}$, $B_{i-1}$, $B_i$, $B_{i+1}$, etc. shown in Figure 3-5. In return, the conductor’s tension depends on the location of those points. As such, an iterative procedure has to be adopted to solve this problem. Assume $dx_i$, $dy_i$ and $dz_i$ are the displacements at the end of the insulator $I$, relative to the end support, (see Figure 3-5b). At the first iteration, ($t=0$), the displacements $dx_i^0$, $dy_i^0$ and $dz_i^0$ are assumed to be equal to zero for all insulators. For an iteration “t”, those displacements will be labelled as $dx_i^t$, $dy_i^t$ and $dz_i^t$. The iterative solution follows those steps:

1. Knowing the location of the conductor’s attachment points, the solution procedure described in sub-section 3.2.4 can be followed to calculate the tension forces in all conductors, i.e. $T_{i-1}$, $T_i$, $T_{i+1}$, etc. Those tension forces vary along the length of each conductor span.

2. The tension at the end of each conductor span can be evaluated. This step will be demonstrated by focusing on insulator (I) as an illustration. In Figure 3-5a, $T_i^t$ and $T_{i+1}^t$ represent the tension in the conductor span left and right of the insulator. The tension force $T_i^t$ can be resolved to three components $RB_{x_i}^t$, $RB_{y_i}^t$ and $RB_{z_i}^t$ in the x, y and z directions respectively. Similarly, the tension force $T_{i+1}^t$ can be resolved to three components $RA_{x_{i+1}}^t$, $RA_{y_{i+1}}^t$ and $RA_{z_{i+1}}^t$. The force transmitted to the insulator, I, can be obtained from an algebraic summation of the tension components as per Eqs. (3-26)-(3-28).

\[
Rx_i^t = RA_{x_{i+1}}^t + RB_{x_i}^t \tag{3-26}
\]

\[
Ry_i^t = RA_{y_{i+1}}^t + RB_{y_i}^t \tag{3-27}
\]

\[
Rz_i^t = RA_{z_{i+1}}^t + RB_{z_i}^t \tag{3-28}
\]

Where $Rx_i^t$, $Ry_i^t$ and $Rz_i^t$ are the components of the force acting on the insulator, I, in the x, y and z directions respectively. A similar procedure can be applied to all other conductor spans and insulators.
3. Those forces can be used to obtain the insulator end displacements for iteration (t+1) as per Eqs. (2-29)-(2-31).

\[ dx_{I}^{t+1} = \frac{R_{x_{I}^{t}}}{R_{res_{I}^{t}}} \]  
\[ dy_{I}^{t+1} = \frac{R_{y_{I}^{t}}}{R_{res_{I}^{t}}} \]  
\[ dz_{I}^{t+1} = \frac{R_{z_{I}^{t}}}{R_{res_{I}^{t}}} \]  

where \( R_{res_{I}^{t}} \) the resultant force at node I; \( R_{res_{I}^{t}} = \sqrt{R_{x_{I}^{t}} ^2 + R_{y_{I}^{t}} ^2 + R_{z_{I}^{t}} ^2} \)

4. The incremental displacement is calculated for the insulator, I, as follows:

\[ \Delta x_{I}^{t} = dx_{I}^{t+1} - dx_{I}^{t} \]  
\[ \Delta y_{I}^{t} = dy_{I}^{t+1} - dy_{I}^{t} \]  
\[ \Delta z_{I}^{t} = dz_{I}^{t+1} - dz_{I}^{t} \]  

This is repeated for all the insulators. The incremental displacement components for all the insulators are compared to a tolerance displacement value \( \Delta \delta \).

5. Steps 1 to 4 are repeated till all the components of the incremental displacements for all insulators become smaller than the tolerance value.

6. The conductor’s tension and geometry can be evaluated for all spans at the convergent solution.

It was noticed that attempting to converge \( \Delta x \), \( \Delta y \) and \( \Delta z \) at the same iteration could result in numerical instability. The reason for this instability is due to the high level of coupling between displacements and reactions in the longitudinal direction. This numerical instability was eliminated by obtaining a convergent solution for \( \Delta x \) first and then iterations can be done to obtain a convergent solution for both \( \Delta y \) and \( \Delta z \).

### 3.4 Validation of Multiple Span Solution

The objective of this section is to validate the developed approach for solving multiple span conductors with flexible supports under downbursts. The downburst wind field utilized in this study is based on Hangen and Kim (2007) and Hangan and Kim (2008)
computational fluid dynamics (CFD) analysis in which the downburst was simulated a downward jet. Shehata et al. (2005) scaled up the CFD data to calculate downburst loading on the conductors. The CFD model produces radial and vertical velocity fields that vary in time and space depending on the jet diameter $D_J$ and jet velocity $V_J$. Accordingly, the response of (TL) under downbursts depends on these parameters and the relative location between the (TL) and the downburst, which is defined by the polar coordinates $R$ and $\theta$. A schematic showing the location of the downburst relative to the (TL) along with the values of those parameters are given in Figure 3-6d. According to El Damatty and ELawady (2018), this downburst configuration is found to be critical for the considered line and can lead to the failure of the intermediate tower. These configurations induce unequal wind loads acting on the conductors located on either sides of the middle tower. The corresponding distribution of downburst transverse load is presented in Figure 3-6c. Six spans are utilized in this study to obtain an accurate prediction for the conductor’s reaction transferred to the tower as recommended by Shehata et al. (2005). The first and last nodes of the considered six-spanned system are assumed to be restrained in three directions. The properties of the chosen conductors are summarized in Table 3-2. Two cases were selected to validate the developed analytical approach. The first case represents a levelled line, whereas all the towers are on the same level and in the same plane as shown in Figure 3-6a. The second case represents the unlevelled line where the tower in the middle has a misalignment of -8.68, -5 and -10 meters in the x, y and z directions respectively as shown in Figure 3-6b.
Figure 3-6 Multiple Conductor spans (a) Case 1, levelled span Conductors (b) Case 2, unlevelled span conductors (c) Transverse load distribution due to downburst loading  (d) Downburst relative location to the (TL)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Length ALx (m)</td>
<td>450.0</td>
</tr>
<tr>
<td>Sag Length h (m)</td>
<td>19.5</td>
</tr>
<tr>
<td>Cable length S (m)</td>
<td>452.25</td>
</tr>
<tr>
<td>Elasticity Modulus E(N/m²)</td>
<td>6.48E10</td>
</tr>
</tbody>
</table>
### 3.5 Analysis Results

In order to assess the accuracy and the efficiency of the developed approach, the same conductor system is reanalyzed using nonlinear Finite Element Analysis (FEA). The commercial software SAP 2000 is utilized to perform the FEA. The results are obtained in terms of the nodal reactions and nodal displacements, which are summarized in Table 2-3 for the levelled case and in Table 3-4 for the unlevelled case. Differences between the responses predicted using the developed analytical approach and those by employing the FEA are also summarized in the two tables. The maximum differences in the displacements are 9% for the levelled case and 10% for the unlevelled case. Meanwhile, the maximum differences in the reactions are 7% for the levelled case and 9% for the unlevelled case. This is considered a good agreement between the analytical approach and the FEA results, providing a validation for the developed technique.

In terms of efficiency, the developed approach shows a significant reduction in the computational time required to perform the analysis when compared with the FEA. The developed approach took only 1.5 seconds to solve the unlevelled six-spanned conductor’s problem, while the FEA took 330 seconds to solve the same problem. This means that the developed approach is about 220 times faster than the FEA. It is worth mentioning that this exercise is often repeated many times by varying the event size, \( D_1 \), and its location (\( R \) and \( \theta \)) in order to obtain the maximum forces acting on a tower due to the downburst event. As such, the savings in the computational time for the entire parametric study is very significant. Furthermore, the mathematical approach provided is much easier for implementation in an in-house developed computer code for the analysis of an entire transmission line system under high intensity wind events.
Table 3-3 Nodal reactions and displacement for the levelled case (case 1)

<table>
<thead>
<tr>
<th>Node</th>
<th>Finite Element Analysis</th>
<th>Analytical Approach</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rx(N)</td>
<td>Ry(N)</td>
<td>Rz(N)</td>
</tr>
<tr>
<td>I-3</td>
<td>-27770</td>
<td>-1</td>
<td>4553</td>
</tr>
<tr>
<td>I-2</td>
<td>-1120</td>
<td>-6</td>
<td>9092</td>
</tr>
<tr>
<td>I-1</td>
<td>-4159</td>
<td>-2710</td>
<td>9166</td>
</tr>
<tr>
<td>I</td>
<td>-6870</td>
<td>-10674</td>
<td>9071</td>
</tr>
<tr>
<td>I+1</td>
<td>3130</td>
<td>-13018</td>
<td>9152</td>
</tr>
<tr>
<td>I+2</td>
<td>4000</td>
<td>-5512</td>
<td>9089</td>
</tr>
<tr>
<td>I+3</td>
<td>32789</td>
<td>-125</td>
<td>4527</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Node</th>
<th>Finite Element Analysis</th>
<th>Analytical Approach</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dx(m)</td>
<td>dy(m)</td>
<td>dz(m)</td>
</tr>
<tr>
<td>I-2</td>
<td>0.32</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>I-1</td>
<td>0.75</td>
<td>0.68</td>
<td>0.22</td>
</tr>
<tr>
<td>I</td>
<td>1.01</td>
<td>1.67</td>
<td>0.98</td>
</tr>
<tr>
<td>I+1</td>
<td>-0.40</td>
<td>1.96</td>
<td>1.04</td>
</tr>
<tr>
<td>I+2</td>
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<td>1.18</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Table 3-4 Nodal reactions and displacements for unlevelled case (case 2)

<table>
<thead>
<tr>
<th>Node</th>
<th>Finite Element Analysis</th>
<th>Analytical Approach</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rx(N)</td>
<td>Ry(N)</td>
<td>Rz(N)</td>
</tr>
<tr>
<td>I-3</td>
<td>-27316</td>
<td>-3</td>
<td>4545</td>
</tr>
<tr>
<td>I-2</td>
<td>1189</td>
<td>-86</td>
<td>9081</td>
</tr>
<tr>
<td>I-1</td>
<td>3646</td>
<td>-3140</td>
<td>9537</td>
</tr>
<tr>
<td>I</td>
<td>-42204</td>
<td>-9080</td>
<td>7348</td>
</tr>
<tr>
<td>I+1</td>
<td>14886</td>
<td>-14111</td>
<td>10525</td>
</tr>
<tr>
<td>I+2</td>
<td>5216</td>
<td>-5285</td>
<td>9063</td>
</tr>
<tr>
<td>I+3</td>
<td>44582</td>
<td>-323</td>
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<table>
<thead>
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<th>Node</th>
<th>Finite Element Analysis</th>
<th>Analytical Approach</th>
<th>Difference %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dx(m)</td>
<td>dy(m)</td>
<td>dz(m)</td>
</tr>
<tr>
<td>I-2</td>
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<td>0.02</td>
<td>0.03</td>
</tr>
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<tr>
<td>I</td>
<td>2.35</td>
<td>0.59</td>
<td>2.03</td>
</tr>
<tr>
<td>I+1</td>
<td>-1.57</td>
<td>1.49</td>
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</tr>
<tr>
<td>I+2</td>
<td>-1.09</td>
<td>1.10</td>
<td>0.55</td>
</tr>
</tbody>
</table>

3.6 Conclusion

An extensible catenary analytical approach is derived and validated in this chapter. The analytical approach provides solution for multi-span cables with vertical and horizontal misalignment of the cable supports. It considers the combined effect of vertical gravity
loads and transverse wind loads. Also, it accounts for the flexibility of the cable supports and the extensibility of the cables. This powerful analytical method can be used in many applications. However, the major drive in developing such an approach is related to problem of transmission line structures subjected to downbursts. The localized nature of those events can lead to the failure of one tower, which results in misalignment of the spans of the conductors adjacent to this tower. The prediction of the conductors tension forces at this stage is important as they cause unbalanced force on the adjacent towers that can lead to a cascade type of failure. The analytical approach was developed in stages. It started from the basic inextensible solution for single span levelled conductors under in plane loading. It progressed to include the effect of cable extensibility and then the effect of vertical misalignment. This is then extended to include the effect of transverse loads and horizontal misalignment. Finally, the solution progressed to account for multiple span cables with flexible supports, which results from the flexibility of the insulators supporting the conductors in transmission lines. The validation of the developed analytical approach is conducted by comparing its prediction to finite element solutions. The validation is done in two sequences. The first one considered a single span conductor, while the second one considered multi-span conductors supported by insulators. Support misalignment and both vertical and transverse loads are considered in both cases. The multi-span example considered a real downburst load case obtained from a previous computational fluid dynamics (CFD) simulation. The multiple spans utilized in this study involved two cases; levelled case and the unlevelled case. The approach predicted almost the same nodal reactions when compared with the nonlinear FEA for the one span analysis; the differences were less than 1%. Meanwhile, it showed good agreement in terms of the predicted reactions and displacements when compared with FEA for analyzing multiple spans. The maximum difference in the displacement between the two methods was 9% for case 1 and 10% for case 2. In terms of the reactions, a maximum difference of 7% was recorded for the levelled case and of 9% for the unlevelled case. The developed analytical method provides a significant reduction in the computational time compared to FEA. The developed approach is 220 times faster than the FEA for the unlevelled conductor spans case. Analysis of transmission lines under downburst events requires conducting computationally intensive analyses to capture the
downburst sizes and relative location to the line. Therefore, there is a significant benefit in reducing the computational time for each analytical case.

3.7 Acknowledgments

The authors gratefully acknowledge Hydro One Inc., Ontario, Canada and the Natural Sciences and Engineering Research Council of Canada (NSERC) for their financial support provided for this research.

3.8 References

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Chapter 4

4 Numerical Model of Cascade Failure of Transmission Lines under Downbursts

4.1 Introduction

Transmission lines are one of the important elements of modern electrical power utilities. The tower-conductor coupling systems are vulnerable to natural disasters due to their extensive lengths. Several studies in the open literature (Hawes and Dempsey (1993), Dempsey and White (1996), McCarthy and Melsness (1996) and Kank et al. (2007)) have reported that these structures are prone to failure under High Intensity Winds (HIW) in the form of downbursts and tornados. HIW events are usually associated with thunderstorms in the form of rising air and descending air masses. The updrafts are formed by warm moist air and downdrafts are formed by colder air. Fujita (1985) and (1990) defined a downburst as a mass of cold and moist air that drops suddenly from the thunderstorm cloud base, impinging on the ground surface and then transferring horizontally. When a tower fails due to high wind, the conductor forces on both sides of the adjacent towers become unbalanced leading to additional longitudinal and transverse loads on these towers. If the adjacent towers cannot withstand these loads, then failure will propagate causing a cascade. Cascades that occur during extreme wind and ice storms are considered to be the major cause of severe transmission line accidents worldwide, (CIGRÈ (2012)). The effect of cascading failures can be devastating as they can result in lengthy and expensive power outages. There have been many cascading tower failures around the world such those reported by Frandsen and Juul (1976), Oswald et al. (1994) and EPRI (1996). Also, several cascade failures of transmission line system under downburst events have been reported in Canada such as the failure of 19 transmission towers located near Winnipeg, Manitoba reported by McCarthy and Melsness (1996).
Following this collapse, an extensive research program has been conducted at the University of Western Ontario. The research started by the development and validation of a computational Fluid Dynamics model (CFD) to simulate the downburst wind field, (Kim and Hangen (2007)). In this simulation, the downburst was modeled as an impinging jet and the solution was obtained using RANS- CFD simulation. Shehata et al (2005) developed a numerical model to simulate the structural behavior of a tower under downbursts. They incorporated the downburst wind field produced by Kim and Hangen (2007). The numerical model evaluated the aerodynamic forces acting on one tower and the attached conductors arising from downbursts. It incorporated a parametric study to evaluate the peak internal forces in the tower members that involved varying the size and location of the downburst. This numerical model was then used to study the behavior of guyed towers (Shehata et al (2005)) and self-supported towers (Darwish and El Damatty (2011)) under downbursts. The numerical model was extended by including failure criteria for the tower members and was used by Shehata and El Damatty (2008) to study the progressive failure of a single guyed transmission tower. The above studies were conducted in a quasi-static manner. Due to the relative high rigidity of the towers (typically >1 Hz) the resonant response is often neglected in the transmission lines analysis. However, conductors are the most sensitive part of the structure due to their flexibility (natural frequency of 0.1-0.2 Hz) that are expected to be prone to dynamic excitation of downburst fluctuating component (frequency of the turbulence > 0.05 Hz in most of the cases). Previous studies such as Darwish et al. (2010), Lin et al. (2012) and Aboshosha et al. (2015) showed that the aerodynamic damping plays an important role in diminishing the dynamic response of the conductors. In a recent study conducted by Ibrahim. et al (2017), a dynamic analysis was conducted to predict the dynamic response of the conductor systems due to downburst loading and to examine the validity of using quasi-static analysis. The study revealed that the conductor system are dynamically insensitive and can therefore be treated quasi-statically. Moreover, the results of the aero-elastic experiment recently conducted by El Awady et al. (2017) at the Wind Engineering, Energy and Environmental (WindEEE) dome showed that the dynamic response of the tower and the conductors is relatively small at the expected high downburst velocities.
In addition to the author’s research group, other researchers conducted the following studies related to the response of transmission lines to downbursts. Mara et al. (2016) assessed the load-deformation curve of a transmission tower under downburst wind loading and compared it to that obtained under a normal wind loading profile. The analysis considered nonlinear inelastic response under simulated downburst wind fields. Yang and Hong (2016) reported the capacity curve of a single tower within the tower-line system under the downbursts considering both the mean and the turbulent wind components and the tower-conductor interaction. The study employed the incremental dynamic analysis and the nonlinear static pushover analysis to estimate the capacity curve. They conducted a comparison of capacity curve of a tower within a tower-line system to that of a single tower to determine the effect of the dynamic interaction between the tower and the conductors on the tower’s capacity.

It is clear that none of the previous investigations considered the behavior of multiple towers and the attached conductors as a system. This is important since the failure of one tower can trigger the failure of the adjacent towers as the towers are connected through the conductors. In this study, a unique numerical model is developed and validated to study the progressive failure of multiple towers of a transmission line segment under downbursts. The numerical model is able to predict the location of the downburst that is the most critical to a specific tower and it can predict the progression of failure of the members of this tower until it fully collapses. The model is also capable of predicting the location at which a mechanism occurs in the failed tower. During the collapse of the tower, the forces in the conductors tend to increase, which can bring the tower to a new state of equilibrium. The numerical model is capable of predicting this new equilibrium state and the conductors forces associated with this state. In addition to the downburst forces, the adjacent towers will be subjected to unbalanced conductor forces due to the change of the tower-conductor location of the collapsed tower. The adjacent towers are then analyzed under the downburst and the unbalanced conductor forces and their resistance to those forces is assessed. As such, the analysis can progress from tower to another along the entire studied segment of the line. Such a numerical model attempts to describe the behaviour and progressive failure of an entire transmission line segment due to downbursts.
The details of various components of the developed numerical model and the validation process of those components are described in the first part of the paper. In the second part of the paper, the numerical model is used to study the progressive failure of a segment of a real transmission line system consisting of nine self-supported towers and the attached conductors as a case study illustrating the capabilities of the numerical model.

### 4.2 Components and Validation of the Numerical Model

The numerical model reported in the current study is based on five main components. In Table 4-1, those components are listed together with the studies in which those components are developed and validated.

<table>
<thead>
<tr>
<th>Component</th>
<th>Developed</th>
<th>Validation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Downburst Wind Field</td>
<td>Kim and Hangen (2007)</td>
<td>The downburst was validated experimentally by Kim and Hangen (2007)</td>
</tr>
<tr>
<td>4. Structural model for progressive failure of a single tower</td>
<td>Current Study</td>
<td>Current Study</td>
</tr>
</tbody>
</table>
4.2.1 Downburst Wind Field

A downburst event is a metrological phenomenon that is localized, random, and hard to measure by traditional recording stations. As such, the majority of researchers studying downbursts loading of transmission line systems used computational fluid dynamic (CFD) - generated wind field to determine the loading on the structure system. Therefore, the simulation of a downburst wind field in this study is conducted numerically based on an impinging jet (CFD) model that was developed and validated by Kim and Hnagen (2007). These CFD models yielded time series for a vertical (axial) component \(V_{VR}\) of the velocity field, as well as a horizontal (radial) component \(V_{RD}\). The velocity fields of these two components vary in time and space, depending on the jet diameter \(D_J\) and jet velocity \(V_J\). Shehata et al. (2005) conducted an extensive parametric study to investigate the downburst wind profiles. Figure 4-1 illustrates the variation of the radial velocity, normalized with respect to the initial jet velocity, along the height. The maximum velocity profile was found to occur at an \(R/D_J\) value of 1.2. The absolute maximum velocity is approximately equal to 1.1 \(V_J\). On the other hand, Figure 4-2 shows the profile of the vertical velocity, normalized with respect to the jet velocity, along the height. The vertical component is quite small compared to the radial component at the first 100 m above the ground, i.e. within the height of typical transmission towers (see Figures 4-1 and 4-2). Furthermore, for the time variation, the time history of the radial velocity component \(V_{RD}\) follows a trend with a maximum peak followed by a minimum peak at which it remains constant throughout the rest of the time history as shown in Figs.4-3.
4.2.2 Basic Structural Model for Single Tower

Shehata. et. al (2005) developed the basic finite element model that can be used to predict the structural performance of a tower as a part of a transmission line system under downburst loading. A two-node linear three-dimensional frame element with three translational and three rotational degrees of freedom per node is used to model the tower members. Each tower member is simulated by one element. Rigid connections are assumed between the tower members as these are physically connected using multi-bolted connections that can transfer moments. Shehata et al. (2005) utilized the CFD model developed by Hangan et al. (2003) and they applied a scaling procedure was
adopted for the CFD data in order to estimate the spatial and time variations of the wind velocities associated with full-scale downbursts. The analysis assumed that the background component of the response was taken into account by scaling up the mean component produced by the CFD analysis to the gust wind speed. This means that contribution of the resonant component to the overall response is assumed negligible. The magnitudes of the velocity components depend on the jet diameter \((D_J)\), and the jet velocity \((V_J)\). They also differ temporally and spatially depending on the distance \((R)\) measured relative to the downburst center. Shehata et. al (2005) concluded that for different jet diameters \(D_J\), the peak radial velocity was constant at the same \(R/D_J\) ratio while the instant time of the peak radial velocity differed with \(D_J\). Therefore, the distance ratio factor \((R/D_J)\) was used to assess the structural behavior of transmission lines under various downburst configurations.

Shehata et al. (2005) validated their model numerically and further validation was later conducted experimentally through the aero-elastic test conducted on a guyed tower using a unique experiment conducted by Elawady et. al (2017) at WindEEE dome testing facility at The University of Western Ontario in Canada. Different downburst configurations were considered in the test and in the validation process. This validation was reported by Elawady et. al (2018) by comparing the aerodynamic forces, and straining actions predicted by the model to those measured during the test. A comparison was first conducted between the base shears and base moments considering both the mean and the peak components. This provided a confidence in numerical evaluation of the aerodynamic forces under such a transient event. The second level of validation involved comparison of strains at different locations in the towers as well as the guys forces. This provided confidence in the accuracy of the finite element model.

### 4.2.3 Conductor’s Model

In the numerical model developed by Shehata et al. (2005), the conductors were modeled using non-linear finite elements. The conductor analysis should take into account their large geometric nonlinear behavior and it should include the flexibility of insulators as well as the sagging and pretension forces. The analysis is conducted in a time history quasi-static manner. This makes the computational time quite significant. As mentioned
earlier, the prediction of the peak responses of a tower to downbursts requires conducting an extensive parametric study by considering a large number of downburst configurations. All that makes the use of nonlinear finite element for the conductors modeling not practical. To solve this issue, Aboshosha and El Damatty (2014) developed an iterative analytical solution to obtain the response of multi-span conductors supported by insulators under both transverse and vertical non-uniform loading arising from downbursts. Aboshosha and El Damatty (2014) validated their analytical solution through comparison with non-linear finite element analysis. Further validation for this analytical solution was done through the aero-elastic tests conducted for a multi-span transmission line system under downbursts by Elawady et. al. (2017) at the WindEEE facility. The tests confirmed the accuracy of the analytical model in predicting the tension in the conductors under various downburst configurations.

### 4.2.4 Structural Model for Progressive Failure of a Single Tower

Shehata and El Damatty (2008) extended the capabilities of their numerical model to be able to conduct failure analysis of a single transmission tower under downbursts. Their model included the nonlinear material effect. Hamada and El Damatty (2015) extended this model to include the geometric non-linear effect. In the current study, both material and geometric nonlinear effects are included to study the failure of a tower under downbursts. Different failure models were accounted for in this model including the (i) members axial capacity governed by the net section capacity in tension and buckling in compression (ii) connection rupture capacity including bearing and shear capacity.

This part is validated by conducting pushover analysis in the longitudinal and transverse directions on one of the existing towers owned by Hydro one, Ontario, Canada. A nonlinear Finite Element Analysis (FEA) was employed in the validation process where the commercial software ABAQUS (2017) was utilized to perform the pushover analysis. A full description of the validation process is provided in Chapter 2.
4.2.5 Numerical Model to Predict the Conductor Forces Post Failure of the Subject Tower

The semi-analytical solution developed by Aboshosha and El Damatty (2014) is limited in its application to levelled spans. This might not be adequate for failure analysis. The progressive failure of the tower members will eventually lead to a formation of an intermediate hinge in the main body of the tower that will result in the formation of a mechanism. As a result, the unstable part of the tower will start to rotate. Consequently, the conductor’s attachment point to the tower will move vertically and horizontally leading to a change in the conductor’s geometry. As such, an analytical procedure was developed by Shehata and El Damatty (2019) using the extensible catenary approach to predict the conductor tension with end conditions that are not aligned both horizontally and vertically while being subjected to its own weight and transverse wind loads. The developed approach accounts for the flexibility of the cable supports resulting from the presence of insulators and the extensibility of the cables.

In order to assess the accuracy of the extensible catenary approach, a validation was conducted by Shehata and El Damatty (2019) where two conductor systems were proposed; a leveled system and unleveled system horizontally and vertically. The same conductor systems were reanalyzed using nonlinear Finite Element Analysis (FEA). Using the commercial software SAP 2000 (CSI Inc. 2016), the results were obtained in terms of the nodal reactions and nodal displacements for the leveled case and for the unleveled case. The responses predicted using the developed analytical approach and the FEA method showed a good agreement, providing a validation for the developed technique.

4.3 Description of the Numerical Model responsible for Predicting the Progressive Failure of TLs

4.3.1 Numerical Model Assumptions

Considerable amount of research effort has been directed to study the behavior of the towers under longitudinal loads for estimating the static and dynamic loads. There have been extensive studies, both analytical (Tucker (2007) and Shen et al. (2011)) and
experimental (Peyrot et al. (1980) and Kempner (1997)), to determine the maximum transient and residual longitudinal loads on towers due to broken conductor loads and component failure. However, these studies were focused to determine impact factors due to sudden rupture of conductors. At the instant of rupture, a huge amount of potential energy will be released to the system that will amplify the unbalanced longitudinal loads on the adjacent structures. Following the rupture instant, the conductor tension begins to drop and then begins to rise to form the first peak that occurs when the insulator swings towards the horizontal. The second peak occurs afterwards when the conductor starts to free-fall and bottom down. The maximum tension in the conductors will occur in either the first or the second peak, (Peyrot et al. (1980)). After the two peaks, the insulator will stabilize horizontally and in a static equilibrium position. The residual static load (RSL) is defined as the loading criteria corresponding to a broken wire condition in the Guidelines for Electrical Transmission Line Structural Loading (ASCE 1991). However, in the current study and from the industrial point of view, the conductors will still be attached to the cross arms post failure the tower. Meanwhile, this failure will not be accompanied with a rupture in the conductors. Moreover, the failure of the suspension tower will cause conductors stretching instead. Therefore, the quasi-static analysis should be sufficient to analyze transmission tower failures with the attached conductors.

4.3.2 Methodology

In section 4.2, the basic components of the numerical model utilized in this study were described. In addition, other developments were conducted in this study in order to establish a comprehensive model capable of predicting the response and the progressive failure of a segment of an entire line consisting of multiple tower and the in-between conductors. This new development is explained in this section by providing the details of various steps conducted in the failure analysis of the line. Those steps are:

1. Extensive parametric is conducted to determine the downburst critical configurations (R, D, and θ) that to initiate failure in one of the towers.

The following steps are repeated for each configuration;
2. Progressive failure analysis for the considered tower.

3. Prediction of the failure of the subject tower including the location of an intermediate hinge and the post failure new equilibrium state.

4. Calculation of conductor’s force post the failure of the subject tower.

5. Progressive failure analysis of the adjacent towers under conductor forces and downburst-generated loads

4.3.3 Parametric Study

Due to the downburst localized nature, a parametric study should be conducted to find the most critical locations relative to the considered tower. The parameters that define the forces acting on a transmission line (TL) located at the vicinity of a downburst are shown in Figure 4-4. These parameters are the jet diameter $D_J$, the location of the center of the downburst relative to the center of the tower defined by the polar coordinates $(R, \theta)$, and the jet velocity $V_J$. As downburst is a transit event, a linear analysis is conducted for each time step for the entire tower. The analysis is conducted under the combined effect of the downburst nodal forces and the conductor’s reactions predicted from the developed technique done by Aboshosha and El Damatty (2014). The instantaneous values of the tower’s member forces are evaluated throughout the downburst entire time history.

![Figure 4-4 Downburst characteristics parameters](image_url)
For a specific downburst configuration, the peak forces acting on each member of the tower during the downburst duration are evaluated. It should be noted that these maximum forces occur at different time values. These analyses are then repeated by varying the downburst parameters. The range of parameters considered to cover all downburst locations that can affect the tower are as follows:

- \(D_j\) = from 500 m to 2000 m using an increment of 250 m.
- \(R/D_j\) from 0 to 2.2 using an increment of 0.2.
- \(\theta\) from 0° to 90° using an increment of 15° (because of the double symmetry).

The absolute peak internal forces for each member of the tower resulting from all conducted analyses are then evaluated together with the associated downburst critical configurations. The peak forces are divided by the members’ capacity to obtain a strength factor “\(\lambda\)’’ for each member, where \(\lambda\) relates the member peak axial force to its capacity. The higher the value of “\(\lambda\)’’, the more critical is the member and the structure to the downburst loading. The downburst configurations leading to large values of “\(\lambda\)’’ for a significant number of tower members are identified. The following steps are conducted for each critical downburst configuration.

4.3.4 Failure Analysis of the Tower of Interest

A nonlinear time history progressive failure analysis is conducted for the considered tower for each critical downburst configuration. The tower members’ capacities are calculated based on the recommendations given in ASCE (2017). The tower members are assumed to totally fail once the member capacity is reached. The capacity of the member is determined by the least member’s axial capacity and the connection rupture capacity; which is measured by the least of the connection bearing capacity and the connection shear capacity. The tower is being solved incrementally and once the member’s capacity is reached in one of the increments, the member is eliminated from the structural stiffens matrix in the subsequent increments. The deformed shape is captured incrementally, and the intermediate hinge formation can be observed when the structure has lost its overall stability.
4.3.5 Post Failure Prediction and New Equilibrium State

The failure of the tower will occur at a certain location at which an intermediate hinge is formed and the portion of the tower above this hinge will become unstable. The rotation of the unstable portion of the tower about the hinge location will increase the tension force at the conductors. This will tend to stabilize the tower again, which can reach a new state of equilibrium. The prediction of the tower geometry at the new equilibrium state is important since it dictates the tension force in the conductors, which will act on the adjacent towers together with the downburst forces and can trigger their failure. As such, two steps are conducted in this part; (i) determine the intermediate hinge location, (ii) estimation of the post failure second equilibrium shape.

4.3.5.1 Intermediate Hinge Location

Figure 4-5 (a-b) show photos of the failure of two different transmission towers. The photos show the location of the hinges about which the top unstable part of the towers rotates. Numerically the location of the hinge can be determined by examining the deformation shape of the tower. Figure 4-6a shows the deformed shape of the tower at different time steps of the downburst time history. At time steps $t_1$, $t_2$ and $t_3$, the structure behaves fully elastic. At time “$t_f$” a hinge is formed and excessive deformations start to occur. The hinge forms at a height “$h_f$” from the ground above which a mechanism is formed. Usually at this time instant, failure of chord member at this location occurs, making the structure unstable. This approach was used to determine the location of the hinge.

4.3.5.2 Post Failure Equilibrium State

Figure 4-6b shows a schematic of the deformed shape of the tower after the formation of the hinge. In this figure, “$\theta_f$” represents the orientation of the unstable part (AB) with the vertical direction, while “$\theta_d$” represents it’s orientation in a horizontal plan (x-y plan). In this figure, $F_x$, $F_y$ and $F_z$ represent the downburst forces acting on the unstable part of the tower in the x,y and z directions, respectively. Meanwhile, $C_{rx}$, $C_{ry}$ and $C_{rz}$ represent the forces acting on the conductors in the x, y and z directions, respectively. The conductor forces will result from two effects:
The downburst loads acting on the conductors,

The change in conductor’s geometry associated with the rotation of the unstable segment and quantified by the angles \( \theta_f \) and \( \theta_d \).

The angle \( \theta_d \) can be evaluated by calculating the direction of the resultant of the horizontal forces acting on the unstable part, i.e. the resultant of \((F_x+C_r x)\) and \((F_y+C_r y)\). The angle \( \theta_d \) is the angle between the resultant and the y-axis.

The angle \( \theta_f \) is obtained by taking moment about the Hinge (point A). The forces \( F_x, F_y \) and \( F_z \) will create a destabilizing moment \( M_{\text{des}} \), while the forces \( C_r x, C_r y \) and \( C_r z \) will create a stabilizing moment \( M_{\text{st}} \). The segment \( \text{Ab} \) will remain unstable as long as \( M_{\text{dis}} > M_{\text{st}} \). The increase of \( \theta_f \) leads to an extension of the conductors and consequently an increase in the conductor forces and the stabilizing moment \( M_{\text{st}} \).

The evaluation of \( C_r x, C_r y \) and \( C_r z \) follows the procedure described in details in Chapter 3 of the thesis. The forces depend on the values of the angles \( \theta_f \) and \( \theta_d \). As such, an iterative procedure is conducted in order to obtain the values of \( \theta_f \) and \( \theta_d \) that make \( M_{\text{dis}} = M_{\text{st}} \), i.e. leads to new state of equilibrium for the portion \( \text{AB} \) of the tower. At this state, the conductor forces acting on the adjacent towers can be also evaluated from the procedure outlined in Chapter 3. It should be noted that all the conductors should be considered when evaluating the new equilibrium state.

Three possibilities can arise regarding the position of the failed segment:

1. \( \theta_f < 180^\circ \), this means that the unstable segment reaches a new state of equilibrium before having a vertical position
2. \( \theta_f = 180^\circ \), this means that the unstable segment keeps rotating that it reaches inverted vertical position (Figure 4-5b).
3. The unstable segment hit the ground and thus become supported at its tip (Figure 4-5a).
Figure 4-5 Self-Supported tower typical failure mechanism

Figure 4-6 (a) Post Failure Tower's Geometry. (b) Tower's Elastic curve
4.3.6  Failure Analysis for the Adjacent Towers

Consider the plan view of a segment of a line shown in Figure 4-7a. The figure illustrates a critical downburst location corresponding to the tower T1 determined from the analysis conducted in section 4.3.1. Figure 4-7b shows a schematic for the time history of the downburst radial velocity acting on tower T1 (usually velocity at 10 m above the ground is used as a reference). The corresponding schematic of the time histories of the radial velocity acting on towers T2, T3, T4 and T5 are provided in the figure. Since the location of the downburst is the most critical for T1, velocities acting on the other towers have less values compared to that acting on T1.

The progressive analysis of the entire segment involve the following steps:

1- Step 4.3.2 and step 4.3.3 are conducted for tower T1, until a full collapse occurs at time $t_f$. The corresponding values for the angles $\theta_f$ and $\theta_d$ describing the full collapse geometry of the tower are evaluated as outlined in section 4.3.3.

2- Tower T2 is analyzed under the effect of downburst forces acting on the towers and the attached conductors. The conductors on both sides of the tower are assumed to be aligned till time “$t_1$”. At this instant, the conductor on the left of T2 will be misaligned vertically and horizontally relative to the conductors on the right of T2. The analysis of tower T2, will take into consideration the unbalanced forces associated with this misalignment based on the analytical method developed in Chapter 3. Assuming the tower T2 fails at time $t_2$ the corresponding values for $\theta_f$ and $\theta_d$ are calculated.

3- Analysis will proceed by considering the adjacent towers following the same approach.
Figure 4-7 (a) Failure Analysis of the adjacent towers. (b) Radial downburst loading on different towers

4.4 Case Study

The developed numerical model is applied to the transmission line system shown in Figure 4-8 to assess the capacity and progressive failure of the line under the downburst wind field. The towers of the line are self-supported and each has a total height of 51.81 meters. The towers have two ground wires and three cross arms carrying six conductors. The properties of the conductors are summarized in Table 4-2. As shown in Figure 4-8, eight conductor spans and seven towers are considered in the analysis. The first and last nodes of the conductors are assumed to be restrained in three directions. The analysis is conducted by initiating the failure of the middle tower, labelled T1. The adjacent towers
in the right spans are labelled T2, T4 and T6 and the adjacent towers in the left spans are labelled T3, T5 and T7 as shown in Figure 4-8. The progressive failure analysis of the line is conducted assuming a jet velocity $V_J = 40$ m/sec. The results of the analysis are presented in the same sequence of the analysis reported in the previous sections.

![Figure 4-8 Transmission line system used in the failure study.](image)

**Table 4-2 Conductor's Properties.**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span Length Lx (m)</td>
<td>450.0</td>
</tr>
<tr>
<td>Sag Length S (m)</td>
<td>19.5</td>
</tr>
<tr>
<td>Elasticity Modulus E(N/m²)</td>
<td>6.48E10</td>
</tr>
<tr>
<td>Weight W(N/m)</td>
<td>28.96</td>
</tr>
<tr>
<td>Facing Area from the wind (m²/m)</td>
<td>0.035</td>
</tr>
<tr>
<td>Drag coefficient C_d</td>
<td>1.0</td>
</tr>
<tr>
<td>Cross sectional Area (m²)</td>
<td>9.86E-04</td>
</tr>
<tr>
<td>Insulator Length v(m)</td>
<td>2.438</td>
</tr>
<tr>
<td>Insulator Axial Stiffness</td>
<td>Rigid</td>
</tr>
</tbody>
</table>
4.4.1 Parametric Study Results

The first step in the analysis is to determine the downburst configuration that are most critical to tower T1. By varying the size and location of the downburst and comparing the members internal forces to the members’ capacities, the three downburst configurations provided in Table 4-3 were identified as the most critical one for tower T1.

Table 4-3 Downburst Critical Configurations

<table>
<thead>
<tr>
<th>Case</th>
<th>D_J (m)</th>
<th>R/D_J</th>
<th>\theta</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>750</td>
<td>1.2</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>500</td>
<td>1.2</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>750</td>
<td>1.4</td>
<td>30.0</td>
</tr>
</tbody>
</table>

For cases 1 and 2, the direction joining the centers of the downburst and tower T1 is perpendicular leading to maximum transverse loads acting on the tower and its attached conductors. While, Case 3 cause large longitudinal force on the tower’s cross arms due to the non-uniform distribution of the radial velocity along the conductors resulting from this oblique configurations. Cases 1 and 2 caused 32 member failures; four of them were main chord members on the leeward side. The failure was associated with the members buckling due to large compression forces. The remaining failed members were diagonal members and cross bracing members. Case 3 caused 28 member failures; one of them is a chord member on the leeward side in the waist level. The progressive failure analysis of the line will be conducted on the first transverse and oblique cases, i.e. case 1 and case3.

4.4.2 Progressive Failure Analysis of the TL (Transverse Case, Case 1)

The variation of the radial velocity along the time history of the downburst event is shown in Figure 4-9. The solid graph presented in the figure showes the radial velocity recorded at the tower of interest, (T1), at 10 m height. Similar plots are shown for towers T2, T4 and T6. This downburst configuration causes symmetric loads with respect to tower T1. As such, T3, T5 and T7 will experience similar velocities as T2, T4 and T6, respectively. The progress of the failure of all the towers can be described through the six
stages shown in Figure 4-9 (denoted as stage I to VI). Detailed description for the behaviour of those stages is provided before.

Figure 4-9 Downburst radial velocity time history

4.4.2.1 Failure Analysis of the Subject Tower, T1.

Figure 4-10 shows the progressive failure stages for T1, buckling initiates in T1’s cross bracing and diagonal members (stage I). Buckling of diagonal members progressed gradually at subsequent load increments (stages II &III). Then, the chord members on the leeward side exceeded their compression capacity in stage IV. The forces were redistributed to the diagonal, which then failed. At stage V, the cross section is not able to withstand the overturning moment created by the loading and a total collapse occurred at this stage. This occurred at a downburst radial velocity of 42 m/sec at T1’s Center. The failure happened at a height of 30m relative to the ground. This is the location where an intermediate hinge occurred making the top portion of the structure unstable. The top portion of the tower will rotate about the hinge causing an increase in the conductor tension, eventually leading to a new equilibrium state for the tower.
As explained in the previous section, the new equilibrium position is defined by two angles $\theta_d$ and $\theta_f$, measured in horizontal and vertical planes, respectively. The angle $\theta_d$ has the same direction as the direction of the resultant of the forces acting on the portion of the tower above the intermediate hinge location. This leads to $\theta_d = 90^\circ$, which is expected at this downburst location relative to the tower T1. Calculations were conducted to calculate the angle $\theta_f$ as outlined in the previous section. It is found that $\theta_f = 180^\circ$. This means that the failed segment of the tower does not become stable again until it is inverted vertically. The location of the intermediate hinge close to the top of the tower has made the increase in the conductor tension not very significant, as such the unstable segment made a full swing till it became stable again.

Table 3-4 shows the conductor reactions at the tower location before and after the failure of tower T1. It should be noted that after the failure refers to the new equilibrium position of T1. Referring to the Figure 4-6, “before failure” represents the reaction at stages I to IV, while “after failure” represents reactions at stage V and beyond till an adjacent tower fails.
### Failure Analysis of Adjacent Towers, T2 and T3.

Because of symmetry, tower T2 and T3 will exhibit similar behaviour. As such, the behaviour of tower T2 will be only explained here. Figure 4-9 shows the time history of the velocity at a height of 10m acting on this tower. The location of the downburst relative to tower T2 can be seen in Figure 4-11b. This configuration represents an oblique case with respect to tower T2 causing downburst load in both the transverse and the longitudinal direction on the tower. The velocity distribution on the conductors shown in Figure 4-11a indicates that the conductors left and right of T2 are subjected to unequal loads. This leads to a net longitudinal force transferred from the conductors to T2. The values of the conductor forces acting on tower T2 can be extracted from Table 4-4; those are the inverse of the reaction values provided in the table. Before stage V, the longitudinal force acting on tower T2 has a value of 8454 N. At stage V and beyond, this value is increased by 13% (due to the change of the geometry and layout of the conductors after failure) to become 10615 N. Again referring to Figure 4-9, at stage V (at
which the tower T1 fails), the tower T2 is subjected to a radial velocity about 30 m/sec, together with the forces transmitted from the conductors (from Table 4-4, post failure of tower T1). T2 was able to resist the forces corresponding to this stage. As shown in Figure 4-6, at stage V, the downburst velocities starts to decrease on tower T1, while it is still increasing at tower T2. The progressive failure of Tower T2 is shown in Figure 4-12. While some diagonal members failed at stage III, IV and V the tower remained stable up to stage V. At stage VI, the downburst radial velocities on the tower reached about 35 m/sec. This together with the conductor forces that can be extracted from Table 3-4 (for the post failure of T1) leads to the collapse of the tower. An intermediate hinge is formed at this stage at an elevation of 30 m (similar to tower T1). The direction of failure in the horizontal plan is found to have an angle $\theta_d = 127^\circ$, see Figure 4-11b.

This value is controlled by the significant longitudinal conductor forces (acting on the –ve x direction). The post failure of tower T2 did not show a full swing similar to tower T1. Instead, tower T2 reached the new equilibrium state at a vertical angle $\theta_f = 25^\circ$.

4.4.2.3  Failure Analysis of Adjacent Towers, T4, T5, T6 and T7.

Table 4-5 shows the conductors reactions before the failure of T1 and after the failure of T2 and T3. Before the failure of T1 corresponds to stage I to stage V. After failure of T4 corresponds to stage VI and beyond. Considering, tower T4, the table shows that the changes in the conductor’s layout after the failure of T2 leads to increase in the reaction Crx (longitudinal) at tower T4 by 168%.

Tower T4, is subjected to the conductor forces together with the radial velocity shown in Figure 4-6. The analysis indicates that the tower T4 was able to survive the entire time history of the downburst without failing. This happened despite that some redundant diagonal members reached their ultimate capacities. Tower T5 exhibited a similar behaviour as tower T4. As such, tower T4 and T5 were able to contain the failure and towers T6 and T7 did not fail. The final failure shape of this segment of the line is shown in Figure 4-13.
Figure 4-11 (a) Downburst transverse loading distribution on the conductors (b) Plan view for the T1 relative to the downburst
<table>
<thead>
<tr>
<th>TOWER</th>
<th>Stage (I-III)</th>
<th>Stage (IV)</th>
<th>Stage (V)</th>
<th>Stage (VI)</th>
</tr>
</thead>
</table>

**T2**

![Diagrams showing progressive failure for T2](image)

**T4**

![Diagrams showing progressive failure for T4](image)

Figure 4-12 T2 and T4 Progressive failure under case 1
Figure 4-13 Case one TL failure mechanisms
Table 4.4 Conductor reactions before and after the failure of T1.

<table>
<thead>
<tr>
<th>Tower No</th>
<th>Before Failure</th>
<th>After Failure</th>
<th>Difference</th>
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<tbody>
<tr>
<td></td>
<td>CRX (N)</td>
<td>CRY (N)</td>
<td>CRZ (N)</td>
</tr>
<tr>
<td>T8</td>
<td>-27001</td>
<td>-73</td>
<td>4553</td>
</tr>
<tr>
<td>T6</td>
<td>1239</td>
<td>-327</td>
<td>9144</td>
</tr>
<tr>
<td>T4</td>
<td>-3504</td>
<td>-2918</td>
<td>9115</td>
</tr>
<tr>
<td>T2</td>
<td>-8454</td>
<td>-10569</td>
<td>9202</td>
</tr>
<tr>
<td>T1</td>
<td>0</td>
<td>-15781</td>
<td>9182</td>
</tr>
<tr>
<td>T3</td>
<td>8454</td>
<td>-10569</td>
<td>9202</td>
</tr>
<tr>
<td>T5</td>
<td>3504</td>
<td>-2918</td>
<td>9115</td>
</tr>
<tr>
<td>T7</td>
<td>1239</td>
<td>-327</td>
<td>9144</td>
</tr>
<tr>
<td>T9</td>
<td>27001</td>
<td>-73</td>
<td>4553</td>
</tr>
</tbody>
</table>

Table 4.5 Conductor reactions before the failure of T1, and after failure of T2 and T3.

<table>
<thead>
<tr>
<th>Tower No</th>
<th>Before Failure</th>
<th>After Failure</th>
<th>Difference</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>CRX (N)</td>
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<td>T6</td>
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<td>1239</td>
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<td>9144</td>
</tr>
<tr>
<td>T9</td>
<td>27001</td>
<td>-73</td>
<td>4553</td>
</tr>
</tbody>
</table>

4.4.3 TL Progressive Failure Analysis (Oblique Case, Case 3)

This case is found to cause high longitudinal force on the tower’s cross arms due to the non-uniform distribution of the radial velocity along the conductors. The corresponding distribution of downburst transverse load is presented in Figure 4-15a. In a view of the parametric study conducted, this is one of the critical cases affecting the tower of interest (T1). However, the adjacent towers (T2 and T4) were more vulnerable than the tower of interest due to their perpendicular location relative to the downburst centre as shown in Figure 4-15b. The angle between T2 and the downburst center is \( \theta = 2.4^\circ \), while T4 is \( \theta = \)
- 22.4°. Therefore, T2 is more vulnerable to the downburst than T4. The TL failure pattern is elaborated in Figure 4-14 and Figure 4-15.

### 4.4.3.1 TL Progressive Failure Stage (I-III)

In stage (I), the failure initiated first in T2 with four diagonal member’s failure on the leeward side and one diagonal member failure in T4, where they exceeded their compression strength. In stage (II-III), three diagonal members failed in T1, nine diagonal members in T2 and six diagonal members in T4. The failed members are presented in Figure 4-14a.

### 4.4.3.2 TL Progressive Failure Stage (IV)

T2 encountered four main chord members failure in the subsequent stage, stage (IV). The chords were located just below the lowest cross arm in the waist level on the leeward side of the tower’s shaft. The chord member’s failure were due to the exceedance of their compression capacities. The chord member failure caused an instability in the upper shaft and a total failure of 40 members, three of them where main chord members. An intermediate hinge was formed at 30 m in height, the failed segment completed a full cycle rotation around an intermediate hinge, $\theta_f = 180^\circ$, and formed an in plane angle $\theta_d$, which is equal to 101° as shown in Figure 4-16b. Which is almost on the same plane connecting the center of the downburst and T2’s center. Meanwhile, T4 experienced 4 member failures, one of them is a main chord member, C3. While T1 experienced 4 diagonal member failures, see Figure 4-14b.
Figure 4-14 TL Cascade Failure for oblique case
Cont. Figure 4-15 TL Cascade Failure for oblique case
Figure 4-16 (a) Downburst transverse load distribution on the conductors (b) Plan view for the oblique case cascaded failure.

4.4.3.3 TL Progressive Failure Stage (V)

As shown in Figure 4-15a, the failure of the main chord member in T4 initiated 15 member’s failures, three of them were main chord members on the leeward side. Accordingly, T4 experienced an intermediate hinge formation at 30 m in height, a full rotation around an intermediate hinge due to the adequate slack in the conductor’s system, $\theta_f = 180^\circ$ and an in plane angle, $\theta_d$, of 86.4° as shown in Figure 4-16b.
4.4.3.4 TL Progressive Failure Stage (VI)

Following the failure of T2 and T4, T1 experienced a failure in one of the chord members, C3, followed by 26 member failures, three of them were main chord members on the leeward side. An intermediate hinge was formed at 30 m in height, a full rotation around an intermediate hinge and an in-plane angle of 82.6° due to the increased tension in the conductor’s. The failure of the T1 tower is shown in Figure 4-15b and the in-plane angle is shown in Figure 4-16b. Although the downburst was in an oblique position relative to the tower of interest, it can be noted that, the adjacent towers T2 and T4 failed first perpendicular to the TL and caused a transverse cascade. This is consistent with what has been found in the open literature that transverse cascades are perpendicular to the line and can be caused by high-intensity winds, Tucker (2007).

4.5 Conclusion

In this Chapter, a unique model is developed to predict the behaviour and the progression of failure that happen from tower to another within a segment of a transmission line during a downburst event. This comprehensive numerical tool consists of various components, which are:

- Downburst wind field based on computational fluid dynamics simulation.
- Numerical model to predict the most critical downburst configuration for a tower and its failure capacity.
- Analytical tool to predict the tension in a conductor supported by unlevelled supports horizontally and vertically.

The models are integrated together in this chapter with a unique capabilities developed to be able to predict the location at which mechanism occurs, the post failure geometry and the new equilibrium state resulting from the increase of the attached conductor forces. The analysis of all the considered towers progress throughout the time history of the downburst taking into account the change in the conductor forces associated with the failure of any tower. The model is then used to simulate a segment of a real transmission line initially designed under a reference gust wind speed of 32 m/sec. The simulation
considered eight spans and seven towers with end conditions of the conductors that are assumed hinges.

The following conclusions can be drawn for the studied case:

1. The parametric study revealed that the most critical downburst configuration is the one associated with a wind field perpendicular to the line \((D_1 = 750, R/D_1 = 1.2, \theta = 0^\circ)\), followed by the oblique case \((D_1 = 750, R/D_1 = 1.4, \theta = 30^\circ)\).

2. For the downburst transverse case, the tower in the middle, T1, is predicted to fail at downburst radial velocity of 45 m/sec measured at T1’s center at 10 m height. An intermediate hinge is formed in the tower’s shaft, 30 m above the ground level, just below the third cross arm in the waist level due to buckling of the two main chords on the leeward side. The failed segment was suspended perpendicular to the line, \(\theta_f = 90^\circ\), and completed a full rotation, \(\theta_d = 180^\circ\), around the intermediate hinge due to the adequate slack in the attached conductors.

3. The first two adjacent towers, T2 and T3, experienced the same failure mechanism because of symmetry. The failure took place in the subsequent increment at downburst radial velocity of 35 m/sec (measured at T1’s center). The conductor’s longitudinal forces increased by 13% due to T1’s failure. An intermediate hinge was formed at 30 m height in the tower’s shaft, the failed segment was suspended with \(\theta_f = 25^\circ\) due to the increased tension in the conductors. The failed segment formed an in plane angle \(\theta_d = 127^\circ\) and \(\theta_d = 53^\circ\) for T2 and T3, respectively.

4. The second two adjacent towers, T4 and T5, did not fail despite an increase in the longitudinal conductor forces by 168% because of the failure of T2 and T3. This is because of negligible downburst loading in this region.

5. The transverse case caused three suspension tower failures and the cascade failure was contained.

6. For the downburst oblique case, a combination of downburst loads in both the transverse and the longitudinal directions on T1’s tower due to non-uniform distribution of radial velocity along the conductors is represented.
7. This case triggered first an intermediate hinge formation at 30 m height in T2’ shaft with $\theta_f = 180^\circ$ and $\theta_d = 82.6^\circ$, followed by T4 failure with an intermediate hinge formation at 30 m height, $\theta_f = 180^\circ$ and $\theta_d = 101^\circ$. In the ensuing increment the middle tower, T1, collapsed with a hinge formation at 30 m height, $\theta_f = 180^\circ$ and $\theta_d = 86.4^\circ$.

8. The oblique case caused three suspension tower failures and the cascade failure was contained.

4.6 Acknowledgment

The authors gratefully acknowledge Hydro One Inc., Ontario, Canada and the Natural Sciences and Engineering Research Council of Canada (NSERC) for their financial support provided for this research.

4.7 References


Chapter 5

5 Parametric Study on the Cascade Failure of Self-Supported Transmission Lines under Downbursts.

5.1 Introduction

In the design of electrical transmission lines, vertical loads induced by gravity and transverse loads induced by wind are the two most dominant loads to be considered. Little attention has been paid to unbalanced longitudinal loads because, generally, the longitudinal forces on either sides of the tower tend to be equal. However, the balance of longitudinal forces could be easily broken because of localized events such as downbursts. Fujita (1990) defined a downburst as a sudden downfall of slow rotating air towards the ground. While reaching the ground, this sudden downfall bursts out violently causing an immediate rise in the wind velocity in the lower region of the ground. In addition, the localized nature of the downbursts might result in a non-uniform and unsymmetrical distribution of the wind loads over the line spans. This results in load cases that do not usually exist under uniform and symmetrical large-scale wind events. Leading to possible failures of some parts of a supporting tower or even the whole tower. As one tower fails, unbalanced longitudinal loads will also be produced on the adjacent towers. If the adjacent towers cannot resist these unbalanced loads, their failures will affect other towers, possibly resulting in an extensive cascading of failures. The damage of cascading failures can be catastrophic. Cascades that occur during extreme wind and ice storms are commonly understood to be the major cause of severe transmission-line accidents worldwide (CIGRE (2012)), see Figure 5-1. Such failures were observed in 1993, 64 mi of 345 kV transmission were lost during an ice storm (Oswald et al. 1994). In 1996, 19 Manitoba hydro towers cascaded because of downbursts in Manitoba, Canada. In 1998, during the North American ice storm, an example of such an infamous cascade failure occurred which left four million people without power for about a week in Quebec, New Brunswick and Ontario. Recently, in 2007 and 2013 Newfoundland has also seen some prolonged power disruptions in many areas.
Considering the critical nature of such structures, tower designers are now mindful of the risk of cascading, but they lack solid failure containment. Building a dead-end tower every ten or so may not be the most economical way to solve the problem. Quantifications of the load produced on one tower by the failure of its adjacent tower is becoming a necessity. While previous studies have considered the effect of downbursts on individual towers (Savory et al. (2001), Shehata et al. (2005), Wang et al. (2009), Darwish and El Damatty (2011), Aboshosha and El Damatty (2012), Mara et al. (2016), Yang and Hong (2016), Elawady and El Damatty (2017)), this study considers for the first time the progressive collapse of an entire line under downbursts.

As such, Shehata and El Damatty (2019a) developed a nonlinear finite element model to capture the incremental deformation and the post-failure geometry of the tower to be used in predicting the forces transmitted through the conductors to the adjacent towers within the line. This novel numerical model allows calculating the conductor’s reactions using three-dimensional extensible catenary mathematical model developed by Shehata and El Damatty (2019b). This is important to predict how the failure of a TL tower within the line will affect the adjacent towers as TLs are designed to function as a unit. The current study builds on the findings of Shehata and El Damatty (2019a). Downburst wind fields, obtained from computational fluid dynamics (CFD) simulations conducted by Hangan and Kim (2007) are incorporated in the fluid-structure program. The numerical model is then used to conduct a parametric study to identify the critical downburst configurations for the self-supported tower. The numerical model is used to; i) predict the cascaded failures of TL systems under the critical downburst loading; ii) assess the performance of an existing TL, designed following current code provisions under the effect of normal wind, when exposed to downburst wind with different jet velocities; iii) investigate the effect of components within the TL (insulator length) on the cascade failure of the TL; iv) investigate the effect of changing the span length on the cascade failure of the TL.
5.2 Numerical model

The current study utilizes the numerical models developed and validated by Shehata and El Damatty (2019a) and Shehata and El Damatty (2019b) to assess the cascade failure of transmission towers and the chain of failures of a segment within the line system under downbursts. Shehata et al. (2005) developed the basic finite element model that can be used to predict the structural performance of a tower as a part of a transmission line system under downburst loading. The tower members are modelled using two nodded three-dimensional frame elements with three rotational and three translation degrees of freedom at each node. Shehata et al. (2005) utilized the CFD model developed by Hangan et al. (2003) and a scaling procedure was adopted for the CFD data in order to estimate the spatial and time variations of the wind velocities associated with full-scale downbursts. Shehata and El Damatty (2008) extended the capabilities of their numerical model to be able to conduct failure analysis of a single transmission tower under downbursts. Their model included the nonlinear material effect. Hamada and El Damatty (2015) extended this model to include the geometric non-linear effect. In the current study, both material and geometric nonlinear effects are included. In addition, different

Figure 5-1 Hydro Quebec cascade failure [Source: http://icestormof1998.tripod.com ]
failure models were accounted for in this model including the (i) members axial capacity governed by the net section capacity in tension and buckling in compression (ii) connection rupture capacity including bearing and shear capacity. The conductor’s behaviour is estimated using the semi-analytical solution developed by Aboshosha and El Damatty (2014) which accounts for the conductor’s geometric nonlinear behavior, the pretension force, the sagging, and the insulator’s flexibility. However, the developed approach is limited in its application to levelled span. Therefore, Shehata and El Damatty (2019) developed an extensible catenary approach that can solve multi-span conductors under in plane and out of plane loading with un-aligned supports horizontally and vertically which arise from the failure of the suspension structures. A full description of the conductor analysis technique is described in Chapter 3. This technique has proven to be computationally efficient compared to typical nonlinear finite element analysis. The utilized numerical models analyze the transmission line in a quasi-static manner. Due to the relative high rigidity of the towers (typically >1 Hz) and the aerodynamic damping of the conductors the resonant response is often neglected in the transmission lines analysis. This assumption coincides with the findings of previous research conducted by Lin et al. (2012), Aboshosha et al. (2015) Darwish et al. (2010) and EL Awady et al. (2017).

5.3 Downburst Wind Field

Because of their localized nature in both space and time, field measurements of downbursts are quite limited. As such, the majority of researchers studying downbursts loading of transmission line systems used computational fluid dynamic (CFD) generated wind field to determine the loading on the structure system. Consequently, the simulation of a downburst wind field in this study is conducted numerically based on a (CFD) model that was developed and validated by Kim and Hnagen (2007).

The resulting downburst velocity has two components: a radial component and a vertical component. The CFD model produced velocity fields of these two components that vary in time and space, depending on the jet diameter \(D_J\) and jet velocity \(V_J\). The current study adopted the scaling procedure proposed by Shehata et al. (2005) for the CFD data in order to estimate the spatial and time variations of the wind velocities associated with full-scale downbursts. The analysis assumed that the background component of the
response was taken into account by scaling up the mean component to the gust wind speed. This means that the resonant contribution to the overall response of transmission line system subjected to downburst winds assumed to be small and therefore neglected. The magnitudes of the velocity components depend on the jet diameter ($D_J$), and the jet velocity ($V_J$). They also differ temporally and spatially depending on the distance ($R$) measured relative to the downburst center. Shehata et. al (2005) concluded that for different jet diameters $D_J$, the peak radial velocity was constant at the same $R/D_J$ ratio while the instant time of the peak radial velocity differed with $D_J$. Therefore, the distance ratio factor ($R/D_J$) was used to assess the structural behavior of transmission lines under various downburst configurations.
5.4 Description of The Self-Supported Transmission Line System

In the current study, the transmission line system belonged to Hydro One, Ontario, Canada is investigated to assess the capacity and failure mechanisms under the downburst wind field. The tower is a self-supported tower and has a total height of 51.81 meters. The tower has two ground wires and three cross arms carrying six conductors. The properties of the conductors are summarized in Table 5-1. The study considered eight span segments, the tower of interest is the tower in the middle and labeled as T1, while the adjacent towers in the right spans are labeled T2,T4,T6. The adjacent towers in the left spans are labelled T3,T5,T7. as shown in Figure 5-2.

![Figure 5-2 Schematic of the studied transmission line](image)

<table>
<thead>
<tr>
<th>Table 5-1 Conductor's Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
</tr>
<tr>
<td>Span Length Lx (m)</td>
</tr>
<tr>
<td>Sag Length S (m)</td>
</tr>
<tr>
<td>Conductor’s Length(m)</td>
</tr>
<tr>
<td>Elasticity Modulus E(N/m²)</td>
</tr>
<tr>
<td>Weight W(N/m)</td>
</tr>
<tr>
<td>Facing Area from the wind (m²/m)</td>
</tr>
<tr>
<td>Drag coefficient Cd</td>
</tr>
<tr>
<td>Cross sectional Area (m²)</td>
</tr>
</tbody>
</table>
5.5 Failure Analysis

The current section reports the progressive failure analysis of the considered transmission line under critical downburst cases with different jet velocities. First, an extensive parametric study is conducted to determine the downburst critical configurations (R, D, and θ) that are likely to initiate failure of a number of critical members of the subject tower. These parameters are shown in Figure 5-3.

Second, a nonlinear time history analysis that involves material nonlinearity using no post yield model is conducted for each critical downburst configuration. The tower is being solved incrementally and once the member’s capacity is reached in one of the increments, the member is eliminated from the structural stiffness matrix in the subsequent increments. The deformed shape is captured incrementally and an intermediate hinge formation can be observed when the structure has lost its overall stability. Third, the failure mechanism is predicted for the entire line including the failed tower segment and post failure geometry of the attached conductors.
The geometry of the collapsed segment can be predicted by investigating its stability under the combination of its own weight, the forces in the conductors due to the downburst loads, and the forces on the nodes of the collapsed segment of the tower as well. This can be achieved through numerical iterations that eventually identifies the geometry that causes a zero-value overturning moment at the location of an intermediate hinge. The analysis is based on the assumption that the conductors remain attached to the failed segment and the adjacent towers. This assumption is based on the observations from the damage surveying reported by the industry. The location of the failed segment is predicted by an intermediate hinge height, \( h_f \), and the two angles (\( \theta_f \) and \( \theta_d \)) as shown in Figure 5-4. In this figure, “\( \theta_f \)” represents the orientation of the failed segment with the vertical direction, while “\( \theta_d \)” represents its orientation in a horizontal plan (x-y plan). Third, using the extensible catenary approach, the tension forces developed in the conductors are calculated based on the post failure geometry. Finally, the adjacent towers are analyzed based on the conductor tension and the downburst-generated forces.

![Figure 5-4 Tower failed segment parameters](image-url)
5.5.1 Effect of Jet Velocity on the Transmission Line Failure Propagation

The failure analysis of the considered transmission line was conducted using different jet velocities (30, 40 and 50 m/sec). The most critical scenarios for the subject tower, T1, resulted from the parametric studies conducted are presented in Table 5-2. Where the first two critical cases are the transverse case where the downburst is perpendicular to the TL and the maximum transverse loads acting on both the middle tower and its attached conductors occur. The table further shows that the critical downburst location occur at R/Dj ratio of 1.2 for all jet diameters, which corresponds with the findings of Shehata and El Damatty (2005). Where peak radial velocities occur at this ratio regardless of the jet diameter’s values. The upcoming failure analysis will therefore focus on case 1 by changing the jet velocities. The variation of the downburst’s radial velocity for different jet velocities along the time history is shown in Figure 5-5. These velocities were recorded at the tower of interest, (T1), under the critical downburst configuration (Dj=750, R/Dj=1.2, θ=0º). The T1 was subjected to a downburst with a jet velocity of 50 m/sec and the collapse of the tower was at a radial velocity of 44 m/sec in the ramping zone.

Table 5-2 Downburst Critical configurations for different jet velocities.

<table>
<thead>
<tr>
<th>Vj</th>
<th>Critical Scenarios</th>
<th>Dj</th>
<th>R/Dj</th>
<th>Theta</th>
</tr>
</thead>
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<td>30</td>
<td>1</td>
<td>750</td>
<td>1.2</td>
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<tr>
<td></td>
<td>3</td>
<td>750</td>
<td>1.2</td>
<td>15</td>
</tr>
</tbody>
</table>

While the same T1 failed at a peak radial velocity of 42 m / sec when exposed to a downburst with a jet velocity of 40 m / sec. It can be observed that the strength of T1 is associated with a downburst radial velocity of 42 m/sec. As a result, no failure is correlated when the tower is exposed to a jet velocity of 30 m/s, see Figure 5-6. The TL
experienced a 14, 10 and 2 diagonal member failures in T1, T2 and T4 respectively and showed no instabilities or failure mechanisms when subjected to jet velocity of 30 m/sec as shown in Figure 5-6a. While, the same tower experienced instability and an intermediate hinge formation right below the lowest cross arm in the waist level at 30 m in height under jet velocity of 40 m/sec. The tower failed at the downburst peak radial velocity, 42 m/sec, the failed segment suspended freely perpendicular to the TL, $\theta_d=90^\circ$, completed a full rotation cycle around the intermediate hinge, $\theta_f=180^\circ$, due to the adequate slack in the conductor’s length.

![Graph](image)

**Figure 5-5 Downburst radial velocity for different jet velocities**

The slack is the difference between the conductor length and the span length. The adjacent two towers, T2 and T3, experienced instabilities and an intermediate hinge formation at 30 m in height and the failed segment suspended towards the middle tower forming an in plane angle, $\theta_d$, which is equal to 127° and 53° for T2 and T3 respectively as shown in Figure 5-6b. However, the main drive behind T2 and T3 failures is the downburst loading on the structure itself and the attached conductors. The failure of the subject tower, T1, was unrelated to the failure of the two adjacent towers. T1’s failure
will impose insignificant unbalanced loads on the adjacent towers due to the sufficient slack in the conductor’s system.

Where the longitudinal conductor’s reactions, $C_{ry}$, increased by 13% for T2 and T3 and by 3% for T4 and T5. The vertical conductor’s reactions, $C_{rz}$, increased only for T2 and T4 by 16%. While the transverse reactions were the same along the entire line. The failure of the suspension towers T2 and T3 caused a significant increase in the conductor’s tension forces on the adjacent towers, T4 and T5. The increase in T4 and T5 conductor’s reactions were estimated by 168%, 4% and 2% in the longitudinal, transverse and vertical directions, respectively. Having said that, T4 and T5 did not experience any structural instability because of the downburst negligible forces in this region. As shown in Table 5-3, the downburst total resultant forces on T4 and T5 towers are 1.4 kN without taking into account the stretched conductors reactions, and 30.3 kN taking into account the stretched conductors reactions. Meanwhile, T6 and T7 experienced 6.6 kN in total due to the downburst loading and the stretched conductors and did not show any kind of failure, even in the members. Similarly, The TL showed almost the same failure mechanisms when subjected to a downburst with a jet velocity of 50 m/sec. Three tower failures in total, T1, T2 and T3. While T4 and T5 experienced higher downburst loads as shown in Table 5-3 but still within the tower’s capacity and shown no instabilities or failure mechanisms as shown in Figure 5-6c.
<table>
<thead>
<tr>
<th>$V_J$</th>
<th>Scenario</th>
<th>ITOWER</th>
<th>$h_f$(m)</th>
<th>$\theta_f$</th>
<th>$\theta_d$</th>
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<td>90</td>
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<td>101</td>
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<td>180</td>
<td>86.4</td>
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<td>1</td>
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<td>90</td>
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<td>180</td>
<td>86.4</td>
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</tr>
</tbody>
</table>
Figure 5-6 Transmission Line Failure mechanisms (a) jet velocity 30m/sec (b) jet velocity 40 m/sec (c) jet velocity 50 m/sec
Table 5-4 Downburst loading on TL towers for the downburst critical case 1 (D_J=750, R/D_J=1.2, θ=0).

<table>
<thead>
<tr>
<th>Tower No</th>
<th>Without Conductors</th>
<th>With Conductors</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>-166505</td>
<td>0</td>
</tr>
<tr>
<td>T2</td>
<td>-97157</td>
<td>40747</td>
</tr>
<tr>
<td>T4</td>
<td>-684</td>
<td>1186</td>
</tr>
<tr>
<td>T6</td>
<td>-42</td>
<td>163</td>
</tr>
</tbody>
</table>

V_J= 50 m/sec

<table>
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<th>Tower No</th>
<th>Without Conductors</th>
<th>With Conductors</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>-186677</td>
<td>0</td>
</tr>
<tr>
<td>T2</td>
<td>-202216</td>
<td>37629</td>
</tr>
<tr>
<td>T4</td>
<td>-30639</td>
<td>23434</td>
</tr>
<tr>
<td>T6</td>
<td>-524</td>
<td>920</td>
</tr>
</tbody>
</table>

5.5.2 Effect of Insulator Length on the Transmission Line Cascade Failure

As the numerical tool can successfully examine the cascade failure of the TL, it can be used to investigate the effect of different components in the TL on the progression of failure within the line. Specifically, the effect of the insulator length is examined by conducting progressive failure analysis of the TL with different insulator lengths varying from 1.44 to 4.44 m. The critical downburst configurations associated with varying the insulator lengths are shown in Table 5-5. Where the transverse case is the predominant case regardless of the insulator length value. The transverse loads will be transferred to the cross arm supports regardless of the insulator length. In addition, the longitudinal forces are negligible due to symmetric downburst loading on the subject tower. However, the oblique case is one of the critical cases for the smaller insulator lengths, 2.44m and 1.44m. This can be justified since the oblique case is found to cause high longitudinal force on the tower’s cross arms due to the non-uniform distribution of the radial velocity along the conductors. For the lengthier insulators, the differential longitudinal force is
compensated with the insulator rotation in the longitudinal directions. Therefore, for higher insulator lengths the oblique case is not one of the critical cases.

Table 5-5 Critical Downburst configurations for different insulator lengths

<table>
<thead>
<tr>
<th>Ins</th>
<th>D_J</th>
<th>R/D_J</th>
<th>θ</th>
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</thead>
<tbody>
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<td>750</td>
<td>1.2</td>
<td>0</td>
</tr>
<tr>
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<td>0</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>1.4</td>
<td>30</td>
</tr>
<tr>
<td>2.44</td>
<td>750</td>
<td>1.2</td>
<td>0</td>
</tr>
<tr>
<td></td>
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<td>0</td>
</tr>
<tr>
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<td>500</td>
<td>1.2</td>
<td>0</td>
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</table>

Regarding the TL failure mechanisms, the previous section revealed that the main drive for the instabilities of the subject tower and the first two adjacent towers, T2 and T3, is the downburst loading on the structure itself and the attached conductors. The failure of the subject tower, T1, was unrelated to the failure of the two adjacent towers. T1’s failure will impose insignificant unbalanced loads on the adjacent towers due to the sufficient slack in the conductor’s system. Therefore, increasing the insulator length more than the design value, 2.44 m, will not affect the failure mechanisms of the adjacent towers. Table 5-6 presents the failure mechanism associated with different Insulator lengths. It’s obvious that the same failure mechanism occurs when increasing the insulator length more than the design value, 2.44 m. However, increasing the insulator length affects the imposed unbalanced loads on the adjacent towers. With a reference insulator length of 2.44 m, if the subject tower fails, it is set to impose 0.85, 0.8, the unbalanced longitudinal loads when using insulator length of 3.44 and 4.44 m respectively. While the failure of the two adjacent towers, T2 and T3, will impose 0.67 and 0.5 the unbalanced longitudinal loads when using insulator lengths of 3.44 and 4.44 m respectively. On the other hand, the length of the insulator has not influenced the unbalanced transverse and vertical loads. Meanwhile, reducing the insulator length will affect significantly the imposed unbalanced loads on the adjacent towers. The failure of the subject tower, T1, will impose 1.6 the
unbalanced longitudinal loads if 1.44m is used as an insulator length rather than 2.44m. While the transverse and vertical unbalanced loads were not affected, see Figure 5-7a. Moreover, the failure of the two adjacent towers, T2 and T3, will impose 2.6, 1.25 and 1 the unbalanced longitudinal, transverse and vertical loads respectively, see Figure 5-7b.

![Figure 5-7 Unbalanced loads ratio vs insulator length (a) on the adjacent towers, T2 and T3, after T1 failure. (b) on the adjacent towers, T4 and T5, after T2 and T3 failure.](image)

In conclusion, increasing the insulator length will reduce the imposed unbalanced loads on the adjacent towers especially the longitudinal component. Table 5-6 presents the TL failure mechanisms for different insulator lengths. As shown in the table, using 1.44m insulator length, the failure propagation extends to 5 towers in case 1, and 4 towers in case 3. However, the propagation was limited only to 3 towers when using 2.44m or longer insulator length. Moreover, the oblique case was one of the critical cases that vanished when using insulator strings lengthier than 2.44m as the insulator rotation in the longitudinal direction will compensate the excessive unbalanced loads.
Table 5-6 TL failure Parameters for different insulator lengths

<table>
<thead>
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<th>Ins</th>
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<th>θd</th>
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<td>90</td>
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<td>1</td>
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<td>52</td>
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</table>
5.5.3 Effect of Span Length on the Transmission Line Failure Propagation

An extensive parametric study is conducted using the numerical model to investigate the effect of the span on the cascade failure of TL towers within the downburst wind field. The parametric study is covering a range of spans from 350-550 m with an increment of 100 m. This range of spans covers the most common spans used in the industry. The tower’s original case is the case highlighted in Table 5-7 where the conductor’s span is 450 and the sag is 19.5m. While changing the span, the percentage of \(\frac{\text{sag}}{\text{span}}\) ratio is kept constant to maintain the same pretension force developed in the conductors. This approach is been used in the industry when utilizing the same tower in different spans as long as the minimum clearance height is satisfied. Moreover, maintaining the same pretension force in this parametric study will help in assessing the effect of changing the span lengths on the unbalanced loads imposed on the adjacent towers together with the tower’s failure mechanisms. Table 5-7 presents the different spans used in the parametric study along with the ground wire’s sag, conductor’s sag and slack.

Table 5-7 Conductor's Geometric Profile.

<table>
<thead>
<tr>
<th>Span</th>
<th>Conductor’s sag</th>
<th>Ground wire’s Sag</th>
<th>Conductor’s Slack</th>
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<td>29.13</td>
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<td>4.11</td>
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The parametric study conducted revealed that the TL with 250 m span experienced one transverse critical downburst configuration. While, TL with 350m span experienced two transverse critical cases. Where the downburst center is perpendicular relative to the TL similar to the 450m first two critical cases. However, TLs with 250 and 350 m span did not experience oblique downburst critical case similar to the 450 m third case. Where, the oblique case is found to cause high longitudinal force on the tower’s cross arms due to the non-uniform distribution of the radial velocity along the conductors. For smaller spans, the developed differential longitudinal loads on the cross arm will be much less.
and within the tower’s capacity. Meanwhile, the 550 m span experienced similar downburst critical cases including the oblique case as shown in Table 5-8.

**Table 5-8 Different Span’s Critical Downburst configurations**

<table>
<thead>
<tr>
<th>Span</th>
<th>Critical Scenarios</th>
<th>D_j</th>
<th>R/D_j</th>
<th>θ</th>
</tr>
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<td>0</td>
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<td></td>
<td>3</td>
<td>750</td>
<td>1.4</td>
<td>30</td>
</tr>
</tbody>
</table>

For the downburst transverse cases, case 1 and 2, The failure mechanisms for all the TLs with different spans, 250, 350, 450 and 550 m are similar. Where T1 forms an intermediate hinge at 30m height, the failed segment is perpendicular to the line and completes a full rotation, except that T1 in the 250 m span doesn’t complete a full rotation, θ_f = 135º, due to the insufficient slack on the attached conductor system. In addition, the next two adjacent towers, T2 and T3, experienced the same intermediate hinge height, 30m, except that the slope with the vertical is 10, 15, 25 and 55 degrees for 250, 350, 450 and 550m spans respectively. Figure 5-8a shows the variation of the failed segment slope with the vertical, θ_f, for the two failed segment in the two adjacent towers, T2 and T3. While the in plane angle keeps increasing with the span increasing due to increased transverse loads and decreased longitudinal loads. The increase in the transverse load is associated to the increase in the conductor’s subjected area to the downburst wind field. While the reduced longitudinal loads is associated to the more slack in the conductor’s system. Figure 5-8b shows the variation of the in plane angle, θ_d, for the failed towers, T1, T2 and T3 for different spans.
<table>
<thead>
<tr>
<th>Span</th>
<th>Scenario</th>
<th>ITOWER</th>
<th>$h_f$(m)</th>
<th>$\theta_f$</th>
<th>$\theta_d$</th>
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5.6 Conclusion

A nonlinear finite element analysis integrated with computational fluid dynamics models has been utilized in the current study to investigate the cascade failure of multiple self-supported TL towers within a TL system under downburst loading. An extensive parametric study is conducted to identify the critical downburst configurations by varying the location of the downburst relative to the studied tower. In view of the critical configurations identified, a progressive failure analysis is conducted for a transmission line system consisting of eight spans and seven towers designed and constructed in accordance with current code provisions to withstand normal synoptic wind loading. The results of the parametric study, together with the progressive failure analysis, are reported for each tower. The results include the critical configuration of downbursts with respect to the structure, associated failure load, and a description of the failure mechanism. The study is then repeated to investigate the difference in the progression of failure in the TL towers under different downburst jet velocities. The study demonstrates that none of the investigated towers can sustain the full load produced by downburst jet velocities more than 40m/sec. However, the failure is contained within the subject tower and the next two adjacent towers. Moreover, the effect of the insulator’s string length on the cascade failure of the towers within the line is investigated. The study concludes that the length of

![Figure 5-8 a transverse case, variation of (a) θd with the span (b) θf with the span.](image-url)
the insulators significantly affects the failure load of the adjacent towers. It can be used to control the propagation of failure under downbursts within one tower among the line. The results of the case study presented in this paper show that increasing the insulator length more than the design value, 2.44m, would reduce the imposed unbalanced loads on the adjacent towers especially the longitudinal component. With a reference insulator length of 2.44 m, if the subject tower fails, it is set to impose 0.85, 0.8, the unbalanced longitudinal loads when using insulator length of 3.44 and 4.44 m respectively. While the failure of the two adjacent towers, T2 and T3, will impose 0.67 and 0.5 the unbalanced longitudinal loads when using insulator lengths of 3.44 and 4.44 m respectively. Meanwhile, reducing the insulator length will affect significantly the imposed unbalanced loads on the adjacent towers. The failure of the subject tower, T1, will impose 1.6 the unbalanced longitudinal loads if 1.44m is used as an insulator length rather than 2.44m. Moreover, the failure of the two adjacent towers, T2 and T3, will impose 2.6 the unbalanced longitudinal loads.

Lastly, a parametric study is conducted to investigate the effect of the span on the cascade failure of the TL towers within the line while maintaining the same pretension forces developed in the conductors by changing the conductor’s sag. The parametric study includes the range of spans within 250 m 550 m. The effect of the span on the failure mechanisms and post-failure geometry is reported. The results show that, for smaller spans 250 m and 350m, the downburst transverse case is the only critical case while the oblique case is not one of the critical cases due to less developed differential longitudinal loads on the towers cross arms. The TL failures were limited to the middle three towers and the cascade was contained. However, the parametric study results show that designing TLs with larger spans increases the resilience of the TL under downbursts. Downbursts has a localized nature and tend to damage the surrounding structures within a certain radius. As the span increases, the adjacent towers exhibits less downburst loading. Moreover, increasing the span leads to more slackness in the conductor’s system. Therefore, the unbalanced loads imposed on the adjacent towers will be reduced.
5.7 Acknowledgment

The authors gratefully acknowledge Hydro One Inc., Ontario, Canada and the Natural Sciences and Engineering Research Council of Canada (NSERC) for their financial support provided for this research.

5.8 References


Chapter 6

6 Conclusions and future work

6.1 Conclusions

The research conducted in this thesis consists of four main parts. In the first part, a nonlinear finite element model is developed to study the response of single towers and to assess their capacities under downbursts. This model accounts for the material and geometric nonlinearity. This model is used to study different existing self-supported and guyed transmission towers. In the second part of the thesis, an extensible catenary approach is developed to analyze transmission line’s conductors with unaligned supports vertically and horizontally under in plane and out of plane loading. The mathematical approach can predict the structural performance of a cable with end points unaligned both vertically and horizontally. The approach takes into account the extensibility of the cable. Moreover, it considers the analysis of a multiple span cable, the flexibility of the cable’s supports and the effect of both in plane and out-plane loads. The developed model can be used in predicting the performance of an entire line post the failure of one tower during a downburst event. In the third part of the thesis, the nonlinear finite element model developed in part 1 was extended along with the extensible catenary approach developed in part 2 to analyze an entire line under downbursts. In the fourth part of the thesis, an extensive parametric study is conducted using the developed numerical model to identify the critical downburst configurations by varying the location of the downburst relative to the studied tower. In view of the critical configurations identified, a cascade failure analysis is conducted for a transmission line system consisting of eight spans and seven towers designed and constructed in accordance with current code provisions to withstand normal synoptic wind loading. The results of the parametric study, together with the progressive failure analysis, are reported for each tower. The results include the critical configuration of downbursts with respect to the structure, associated failure load, and a description of the failure mechanism. The study is then repeated to investigate the difference in the cascade of failure in the TL towers under different downburst jet velocities, investigate the effect of components within the TL (insulator length) on the
cascade failure of the TL, investigate the effect of changing the span length on the
cascade failure of the TL.

The following are the main conclusions from the study of progressive failure analysis of
transmission towers under downbursts.

1. The load configuration of \((D_J=750, \theta=90^\circ, R/D_J = 1.3)\) results in maximum
longitudinal loads on the transmission towers. This load configuration, which does
not induce loads on the conductors, is found to be critical for the guyed tower G2.

2. The load configuration of \((D_J=750, \theta=0^\circ, R/D_J = 1.3)\) causes the maximum transverse
loads on the line. This load configuration is found to be critical for the self supported
towers (S1, S2) the guyed tower G1.

3. An intermediate hinge is formed just below the lowest cross arm in the self-supported
towers on the leeward side, where maximum compression stresses occur in the chord
members. S1 exhibits local failure mechanism just below the lowest cross arm ; S2
exhibits a global failure mechanism starting from the lowest cross arm and
propagating to the base because of the cantilever action.

4. The guyed tower G1 intermediate hinge is formed in the guy’s cross arm and is
propagating in the tower’s shaft members. G2 intermediate hinge is formed in the
upper two thirds of the two truss columns, below the cross arm.

5. The weight needed for the self-supported towers to withstand a downburst jet velocity
of 30, 40 and 50 m/sec is 2%, 15% and 42%, respectively, for S1 and 0%, 4% and
12%, respectively, for S2.

6. The weight needed for the guyed towers to withstand a downburst jet velocity of 30,
40 and 50 m/sec is 0%, 5% and 10%, respectively, for G1, and 0%, 10% and 25%,
respectively, for G2.
The main conclusions from the case study of cascade failure of self-supported transmission lines under downbursts:

1. The most critical downburst configuration is the one associated with a wind field perpendicular to the line, $\theta=0^\circ$, followed by the oblique case, $\theta=30^\circ$.

2. For the transverse case, the tower in the middle is predicted to fail due to the maximum transverse loads associated with the perpendicular downburst. An intermediate hinge is formed in the tower’s shaft just below the third cross arm in the waist level due to buckling of the two main chords on the leeward side. The failed segment was suspended perpendicular to the line and completed a full rotation around an intermediate hinge due to the adequate slack in the attached conductors.

3. For the transverse case, the first two adjacent towers, T2 and T3, experienced the same failure mechanism but the failed segment was towards the middle tower’s failed shaft. The second two adjacent towers, T4 and T5, experienced failure in some of their diagonal members but did not lose their overall stability. The downburst caused three suspension tower failures and the cascade failure was contained.

4. The oblique case was expected to cause longitudinal cascade due to the high longitudinal loads on the middle tower associated with the non-uniform distribution of radial velocity along the conductors, But first it triggered a transverse failure on the adjacent towers, T2 and T4, and then in the ensuing increment the middle tower, T1, collapsed. The downburst caused three suspension tower failures and the cascade failure was contained.

6. The results of the case study presented in this paper show that increasing the insulator length more than the design value, 2.44m, would reduce the imposed unbalanced loads on the adjacent towers especially the longitudinal component. With a reference insulator length of 2.44 m, if the subject tower fails, it is set to impose 0.85, 0.8, the unbalanced longitudinal loads when using insulator length of 3.44 and 4.44 m respectively. While the failure of the two adjacent towers, T2 and T3, will impose 0.67
and 0.5 the unbalanced longitudinal loads when using insulator lengths of 3.44 and 4.44 m respectively.

7. Reducing the insulator length will affect significantly the imposed unbalanced loads on the adjacent towers. The failure of the subject tower, T1, will impose 1.6 the unbalanced longitudinal loads if 1.44m is used as an insulator length rather than 2.44m. Moreover, the failure of the two adjacent towers, T2 and T3, will impose 2.6 the unbalanced longitudinal loads.

8. The results show that, for smaller spans 250 m and 350m, the downburst transverse case is the only critical case while the oblique case is not one of the critical cases due to less developed differential longitudinal loads on the towers cross arms.

9. However, the parametric study results show that designing TLs with larger spans increases the resilience of the TL under downbursts. Downbursts has a localized nature and tend to damage the surrounding structures within a certain radius. As the span increases, the adjacent towers exhibits less downburst loading. Moreover, increasing the span leads to more slackness in the conductor’s system. Therefore, the unbalanced loads imposed on the adjacent towers will be reduced.

**6.2 Future work**

The current study discussed the quasi-static responses of self-supported transmission lines due to the downburst loads assuming an open terrain. For future research, the following investigations are suggested:

1. Predict the progressive collapse and chain of failures of guyed transmission towers along a line subjected to downbursts.

2. Expand the extensible catenary model developed in chapter 2 to account for downburst variable loads along the span.

3. Expand the study conducted in chapter 4 to account for the effect of different terrains.

4. Assess the dynamic behavior of the cascaded transmission towers under downbursts.
5. Expand the numerical model to account for different towers within the same line.

6. Study the progressive failure of large span transmission towers along a line subjected to downbursts.
Curriculum Vitae

Name: Ahmed Shehata

Post-secondary Education and Degrees:

Faculty of Engineering, Cairo University Cairo, Egypt 2007-2012 B.Sc.

Faculty of Engineering, Cairo University Cairo, Egypt 2012-2015 M.Sc.

The University of Western Ontario London, Ontario, Canada 2016-2020 Ph.D.

Honours and Awards:

Western Graduate Research Scholarship 2016-2020.

Related Work Experience:

Teaching Assistant
The University of Western Ontario 2016-2019

Instructor
The University of Western Ontario 2019-2020

Publications:

