INTERIM REPORT

I-10 MOBILE RIVER BRIDGE

MOBILE, AB

CLIMATOLOGY AND WIND DESIGN

RWDI #2302137

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SUBMITTED TO

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EXECUTIVE SUMMARY

Rowan Williams Davies & Irwin Inc. was retained by Kiewit Engineering Group Inc. to provide wind consulting studies for the proposed I-10 Mobile River Bridge in Mobile, Alabama. This summary presents the results of the wind climate study completed for the bridge:

- A wind climate study has been performed to establish the wind speeds for the design and verification of the aerodynamic stability of the bridge. Per the draft Technical Provisions for the project, a risk category III design wind speed associated with a 1700-year return period has been established for AASHTO LFRD 9th Edition Strength III wind load calculations.

- The wind speed with a 3.1% probability of exceedance in a 3-year construction period was established for wind load calculations for the bridge during construction. This wind speed corresponds to a 97-year return period.

- Wind speeds for aerodynamic stability verification were based on the 1,000 and 10,000-year return periods for the bridge under construction and the completed bridge, respectively.

In addition to the climatology study, the wind design sections of this report discuss the wind tunnel testing program and the wind design methodologies in accordance with the project requirements.

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1 INTRODUCTION

Rowan Williams Davies & Irwin Inc. (RWDI) was retained by Kiewit Engineering Group Inc. to perform comprehensive wind consulting studies for the proposed I-10 Mobile River Bridge in Mobile, Alabama.

This report presents the background, objectives, results, and recommendations for the wind climate study, and will be expanded as additional studies are completed.

1.1 Project Description

The project consists of a new crossing of the Mobile River with a deck-level height of approximately 230 ft above the water. The cable stayed bridge will be symmetrical around the bridge center line and will be supported by two 250-ft tall diamond pylons.

1.2 Objectives

The two objectives of this study were:

1. to determine site-specific design wind speeds and turbulence properties for strength design of the bridge as well as stability verifications based on historical surface measurements as well as hurricane simulation for Mobile, Alabama; and,
2. To discuss the wind tunnel testing program and the wind design methodologies in accordance with the project requirements.

Objective 1 is discussed in Chapter 2 and Objective 2 is discussed in Chapters 3 and 4.

2 WIND CLIMATE ANALYSIS

The wind climate analysis described below has been conducted in accordance with Section 26.5.2 of ASCE 7-16. The definitions of the wind speeds for design and aerodynamic stability verification were based on the following reference documents, codes and standards:

- The draft Technical Provisions for the I-10 Mobile River Bridge Project (dated December 15, 2022);
- The Alabama DOT Structural Design Manual which references AASHTO LRFD 9th Edition;
- AASHTO GSWLB-1, Guide Specification for Wind Loads on Bridges During Construction; and,
- ASCE-7 2016.

2.1 Climate Data

The wind statistics used to determine the design wind speeds, directionality at the bridge site and minimum onset wind speeds for flutter and galloping for the I-10 Mobile River Bridge were based on a combination of measured and modelled wind speeds. The measured wind speeds were taken from the meteorological station located at the Mobile Downtown Airport from 1997 to 2022. Figure 2-1 shows the location of the proposed bridge and airport meteorological station.

The modelled wind speeds were based on a Monte Carlo simulation of tropical cyclone/hurricane wind speeds.

A data quality review of the high wind speeds in the historical records from Mobile Downtown Airport was conducted to ensure that all high wind speeds included in the records were true wind events. Any erroneous data discovered were removed from the dataset so as not to skew the subsequent analyses. Additionally, any hurricanes within the historical record from Mobile Downtown Airport have been filtered out so as to not double count their impacts on the resulting wind climate model.

2.1.1 Modeled Wind Data – Hurricane Winds

As mentioned above, the measured data from Mobile Downtown Airport were augmented by a hurricane simulation provided by Applied Research Associates (ARA). The hurricane simulations were generated using the Monte Carlo Simulation Technique. ARA provided data of hurricane passages both at the surface and upper levels, corresponding to 33 ft and 2000 ft heights, respectively. Based on the height of the bridge deck at 230 ft, the surface level hurricane simulation was used for this study. The hurricane study is based on simulations of 300,000 storm years occurring in the North Atlantic basin and considers all storms that came within a 150-mile radius of the project site.

2.2 Impact of Upwind Terrain at the Bridge and Meteorological Station

The effect of the upwind terrain surrounding the anemometer at Mobile Downtown Airport and the bridge site was accounted for using the ESDU methodology described in Appendix A1 on a direction-by-direction basis. This method was used to transpose the winds at the airport to the bridge site. By taking into consideration the upwind terrain profiles (on a direction-by-direction basis) at the airport anemometer, the measured wind records have been scaled up to gradient height, which is the height above which the roughness/terrain of the surface of the earth no longer slows down/impacts the wind, establishing the regional, independent of terrain wind condition. This gradient wind model can then be readily transposed to the bridge site. The gradient wind model has then been scaled down to the deck height of the bridge on a direction-by-direction basis, based on the terrain conditions at the bridge site. Figure 2-2 illustrates this method of how measured wind speeds are translated from the anemometer locations to the bridge site at deck level.

2.3 Wind Climate Analysis

2.3.1 Regional Wind Climate

The extreme wind climate was assessed by applying two separate models to the historical and modelled data. An extreme value analysis model was used to assess the relationship between wind speed and return period, as described in Appendix A2. A Weibull model and the upcrossing method were used to assess the directionality of the extreme wind climate at the bridge site, described in Appendix A3. Figure 2-3 shows the Durst curve, which was used to convert the mean wind speeds of the wind climate model to 3-second gust wind speeds, so a direct comparison could be made between the wind climate model and ASCE 7-16 and AASHTO LRFD 9th Edition code values. According to the combined extreme value analysis, the 3-second gust basic wind speed for this region is 150 mph for Risk Category II and 161 mph for Risk Category III. These wind speeds are slightly lower than the corresponding wind speeds for the bridge location in AASHTO LRFD 9th Edition and ASCE 7-16. The design wind speed in AASHTO LRFD 9th Edition is a 700 year wind speed of 154 mph, which matches the Risk Category II wind speed in ASCE 7-16. There is no Risk Category III wind speed in AASHTO LRFD 9th Edition; the Risk Category III design wind speed in ASCE 7-16 is 164 mph. Figure 2-4 shows the wind speeds in open terrain at an elevation of 33 ft based on the statistical analyses of the meteorological data from Mobile Downtown Airport, the hurricane simulations, and the code derived wind speeds. The proposed curve for design wind speeds and flutter verification is based on the combined (extra-tropical + hurricane winds) curve.

2.3.2 Wind Speeds at the Bridge Site

The ESDU analysis described in Appendix A1 and Section 2.2 allow for the mean wind profile under strong winds to be determined at the bridge site. This mean wind speed profile is applied in conjunction with the results of the extreme value analysis illustrated in Figure 2-4, to determine the relationship between mean hourly wind speed and return period at the deck elevation of 230 ft. The resulting bridge deck-level wind speeds at the bridge site have been presented in Figure 2-5a and Figure 2-5b and summarized in Table 2-1a and Table 2-1b for winds from 140º and 320º, respectively.

For the design of the completed bridge a 1700-year return period wind speed of 161 mph should be considered, based on Section 13.3.1.5 of the draft Technical Provisions, which specifies that the risk category for the wind analysis should be Risk Category III.

For design during construction, the AASHTO Guide Specifications for Wind Loads on Bridges During Construction (GSWLB-1) has been referenced. Table C4.2.1-1 of GSWLB-1 lists the applicable return periods based on construction duration (N), probability of exceedance during the construction period and load factor. For example, for a construction period of up to 3 years and probability of exceedance of 3.1%, a return period of 97 years is applicable for a load factor of 1.0. This matches the same load level as earlier versions of AASHTO LRFD where construction loads were provided at a return period of 20 years.

To ensure the aerodynamic stability of the bridge, a 10-minute mean wind speed is used to account for the time required for aerodynamic instability to build-up. To reduce the probability of occurrence of an aerodynamic instability, the 10,000-year return period is recommended which corresponds to wind speed of 150 mph (for winds from 140º) and 134 mph (for winds from 320º) at deck-level height of 230 ft, which includes a 9.6% reduction due to the directionality of the extreme wind climate. There is evidence that flutter instability is essentially a function of the wind velocity component normal to the span. Therefore, directionality reduction factors may be applied to the wind speeds for stability assessment of the bridge, which is in accordance with current medium and long span bridge design practice in North America. For the bridge under construction, 1,000-year return period wind speed of 129 mph (for winds from 140º) and 115 mph (for winds from 320º) is recommended at deck-level height of 230 ft, which also include a 9.6% reduction due to the directionality of the extreme wind climate. The directionality of the extreme wind climate is presented in Figure 2-6, as determined via the Weibull models and upcrossing methods described in Appendix A3.

2.3.3 Turbulence Properties at the Bridge Site

The revised ESDU methodology with a direction-by-direction assessment of upwind terrain was used to determine the wind speeds, turbulence intensities and length scales at deck height and arch heights for the site. The equivalent roughness lengths, power law profile (α), turbulence intensities (I_u, I_v, I_w) and length scales (${}^xL_u$, ${}^xL_w$, ${}^yL_u$, ${}^yL_w$, and ${}^zL_w$), which are most important for the bridge buffeting response to strong winds, are given in Table 2-2.

2.4 Wind Climate Analysis Summary

The design wind speeds resulting from the wind climate and site analysis for the I-10 Mobile River Bridge are summarized in Table 2-1a and Table 2-1b for winds from 140º and 320º, respectively. These are wind speeds at deck elevation of 230 ft. The resulting turbulence properties are shown in Table 2-2. The mean-hourly speeds are recommended for structural design of the bridge, and the 10-minute mean speeds are recommended for flutter stability evaluations both during construction and for the completed bridge. The level of risk accepted for the design of the original bridge replacement was maintained for the current design in the establishment of the design wind speeds for the completed bridge.

3 WIND TUNNEL TESTING PROGRAM

3.1 Wind Tunnel Program Overview

The wind tunnel testing program plays an integral role in the overall wind consulting services provided by RWDI. The proposed wind tunnel testing program involves the following three studies, which will each be described in detail in subsequent sections.

1. **Static test of the free-standing tower** to measure the mean wind loads acting on the tower and to derive corresponding static force coefficients.
2. **Sectional model test of the deck** to assess the aerodynamic stability of the deck cross-section and to measure static force and moment coefficients and aerodynamic derivatives. If aerodynamic instabilities are observed during testing that do not meet project criteria, mitigation strategies will be tested. In addition to the completed bridge deck, the bridge deck while under construction will also be tested.
3. **Aeroelastic model test of the full bridge** to assess the aerodynamic stability of the entire structure and to measure the buffeting response of the bridge for validation of the design wind loads. In addition to the completed structure, aeroelastic tests of two critical construction stage (e.g., free-standing tower and cantilever deck stage) as selected by the design team will also be investigated.

3.2 Free-Standing Tower Static Test

3.2.1 Objectives

The objective of the free-standing tower static test is to derive mean aerodynamic force coefficients for the tower to be used as inputs in the derivation of design wind loads.

3.2.2 Modelling and test procedure

A model scale for the free-standing tower test will be selected based on geometric constraints of the wind tunnel, target sit turbulence properties and instrumentation constraints. A rigid scale model will be constructed and mounted to sensitive load cells such that the mean aerodynamic forces and moments acting on the tower can be measured. The model will be segmented in such a way that the forces acting on each tower leg can be measured independently. Due to the rigid nature of the model, this test will not provide a direct quantification of dynamic load effects.

The tower model will be connected to three independent 6 degree-of-freedom load cells near the base of the model such that the shear forces and overturning moments on each of the legs and the overall tower will be measured. Depending on the symmetry of the pylon design, an appropriate wind direction range will be selected over which to perform tests. Measurements will be performed with a wind direction resolution of 10º or less.

Through analysis of the forces/moments measured at the base, distributions of aerodynamic force coefficients for the tower that are consistent with the overall measurements will be derived. The distribution of force coefficients for the tower will then be used as inputs to a buffeting response analysis.

3.3 Deck Sectional Model Test

3.3.1 Objectives

The objectives of the sectional model wind tunnel test of the bridge deck are as follows:

- Measure the wind-induced dynamic response of the deck over a range of wind speeds up to and exceeding the stability criterion wind speed, expressed as a 10-minute mean, to establish whether the deck experiences any aerodynamic instabilities.
- If aerodynamic instabilities are observed that do not meet project criteria, mitigation options will be tested to find an aerodynamically stable section.
- Static aerodynamic force and moment coefficients and aerodynamic derivatives will be measured for the deck section(s) necessary for the derivation of design wind loads.
- Conduct the above for both the completed deck section and the section while under construction. For the two configurations, the stability criterion wind speeds will be defined by the 10,000-year and the 1,000-year return period wind speeds, respectively.

3.3.2 Modelling and test procedure

The sectional model of the bridge deck will be built from aluminum, steel, brass, 3D printed and wood elements to replicate accurately the geometry and distribution of mass of the deck section at model scale. The sectional model will be designed to simulate the mass and mass moment of inertia (MMI) per unit length of the deck based on the provided full-scale mass/MMI properties. An appropriate center-of-mass location will also be respected based on provided information at full-scale. The equivalent mass and MMI are used to include the effects of other components of the bridge in addition to the deck (e.g., cables and towers) that are in motion for a given mode of vibration. The definitions of the equivalent vertical mass and equivalent MMI for the *j*th mode, m_{eqv_j} and I_{eqv_j}, are:

$$\mathrm{M}_{gen_j} = \int_0^S \left[\mu(s)\left(\Phi_{\mathrm{j,x}}^2(\mathrm{s}) + \Phi_{\mathrm{j,y}}^2(\mathrm{s}) + \Phi_{\mathrm{j,z}}^2(\mathrm{s})\right) + I_{xx}(s)\Phi_{\mathrm{j,xx}}^2(\mathrm{s})\right]ds, \tag{3-1}$$

$$m_{eqv_j} = \frac{\mathrm{M}_{gen_j}}{\int_0^L \Phi_{\mathrm{j,z}}^2(\mathrm{s})ds}, \tag{3-2}$$

$$I_{eqv_j} = \frac{\mathrm{M}_{gen_j}}{\int_0^L \Phi_{\mathrm{j,xx}}^2(\mathrm{s})ds}, \tag{3-3}$$

where Φ_j is the mode shape, s is a coordinate along the structure, S is the length of the entire structure, L is the length of the deck, μ is the mass per unit length and I_{xx} is the MMI per unit length. The equivalent mass/MMI are representative of the amount of mass that the deck must displace due to an aerodynamic instability. The equivalent mass is a more representative measure of the mass that a correlated force must displace. The target modes of vibration for the selection of equivalent mass and MMI will be the 1st order vertical and torsional modes, respectively.

The bridge deck model will be mounted in RWDI's state-of-the-art sectional model test rig. It consists of an inner frame suspended by springs mounted to an outer frame capable of rotating the model through a wide range of angles of attack. The model is mounted on shafts passing through a torsional bearing system and allows for the vertical and torsional motions to be simulated independently. The response of the bridge is measured by means of non-contact laser displacement transducers. The drag, lift, and moment loading acting on the bridge section are measured with high precision load cells. Damping can be added to the system by magnetic eddy current damping devices installed on the rig frames. These devices allow the structural damping for vertical and torsional motions to be adjusted independently as desired and to match the expected full-scale values based on the deck construction. All of the components of the test rig are shielded in removable walls which are placed in RWDI's 24-foot wide Irwin wind tunnel to create an ideal 8-foot test section with nearly 2-dimensional flow.

3.3.2.1 Aerodynamic stability

Prior to each free-vibration wind tunnel test, the model will be excited independently in its vertical and torsional degrees of freedom. The free-vibration decays of the model will be recorded and analyzed to set the vertical and torsional frequencies and damping. At each wind speed, the test sequence will be as follows:

1. Set the wind speed and let it stabilize.
2. Excite the sectional model in vertical and torsional motions to ensure no motion-dependent instabilities are missed. Alternatively, this excitation may represent a particularly large gust or other short-duration excitation force.
3. After this initial excitation, the response of the model is observed until a steady state amplitude is achieved.
4. The response of the model and the wind speed in the wind tunnel are then recorded for 60 seconds.
5. Increase the wind speed and repeat steps 2 through 4 until a divergent response such as flutter or galloping is observed or the wind speed is well beyond the 10,000-year, 10-minute mean wind speed.
6. After completion of the test, the damping and frequency are once again measured with a free-vibration decay of the deck sections for each degree of freedom to confirm no significant change of the test conditions occurred over the test.

RWDI's approach for free vibration testing for aerodynamic stability is to test the bridge at angles of attack to the wind of 0° and ±2.5°. The non-parallel winds are intended to capture any instabilities in the atmospheric boundary layer that could lead to non-zero angles of attack. These angles of attack can lead to worse aerodynamic stability. However, due to the unlikely occurrence of high-winds blowing at non-zero angles of attack, RWDI applies a 20% reduction to the stability criterion wind speed for these angles of attack.

3.3.2.2 Aerodynamic derivatives

The same free-vibration test setup will be used for the extraction of the aerodynamic derivatives of the deck section(s). These coefficients will be used to examine the aerodynamic stability of the bridge in more detail and to describe the aerodynamic stiffness and damping in the buffeting response analysis. However, instead of recording the steady state motion, the model is given an initial displacement in both vertical and torsional degrees of freedom and the subsequent decay of the motion is recorded. A baseline condition is established by extracting the frequency and damping from these decays without any wind. Following the baseline test the wind speed is increased and the same decay measurements are made. By carrying out this process over a range of wind speed, the aerodynamic damping, stiffness, and coupling effects can be quantified using aerodynamic derivatives.

3.3.2.3 Static Force and Moment Coefficients

To measure the static force and moment coefficients, all moving parts of the suspension rig will be mechanically locked out. The bridge model will be mounted on either side to two sensitive shafts that are instrumented with strain gauges to measure the drag, lift, and moment acting on the model. These forces will be measured at high wind speeds over a range of angles of attack from -10° to +10° in 2° increments.

3.3.2.4 Wind buffeting

While the sectional model test is not used to directly quantify the buffeting response of the bridge deck, the extracted information such as the force and moment coefficients and aerodynamic derivatives will be used in RWDI's structural response solver to quantify these effects.

3.4 Full Bridge Aeroelastic Model Test

3.4.1 Objectives

The objectives of aeroelastic model wind tunnel tests are to:

- Assess the aerodynamic stability of the entire structure,
- Measure the wind induced responses to turbulent winds for validation of the design wind loads and directionsand,
- Conduct the above for the completed bridge in service, the free-standing tower construction stage, and one additional critical construction stage to be determined in consultation with the design team.

3.4.2 Modelling and test procedure

The aeroelastic model will be designed using structural information as well as dynamic properties provided by the design team and based on the similarity principles for structural dynamics and aerodynamics (Ref 1, 2). The resulting model will be a lumped mass, dynamic model where the stiffness properties of the various structural components such as the deck, cables, towers and piers are provided by metallic "spines" within the model. The exterior geometry of the structure will be modelled using "shells" that are connected at discrete locations along the structural spines. These shells will be built in segments using a combination of stereolithography (a form of 3D printing) and hand fabrication. The shells give each portion of the structure its external shape and aerodynamic characteristics while allowing the spines of each segment the ability to flex and respond similarly to the full-scale structure. The shells will also provide the additional mass and mass moment of inertia (MMI) that is needed in addition to the spine to reach the target values. In doing so, the distributions of mass and MMI along the structure will be accurately simulated. The length of individual shells will be selected to provide a proper distribution of the aerodynamic forces and moments, while also providing adequate spatial resolution of important mode shapes. The stay cables will be modelled using a combination of thin steel wire and springs that provided the equivalent stiffness of the full-scale cables. Due to the scale of the model, the wires themselves typically cannot practically achieve the scaled stiffness of the cables, therefore springs with the correct stiffness will be added between the towers and the wires. To match properly both the cable mass and the drag force acting on the cable, dowels of finite length will be added to the wires. The number, length and density of the dowels will be selected to achieve the correct cable mass and a drag force that matches the full-scale drag at the design wind speed.

The design of the model will respect Froude number scaling principles (i.e., ratio of wind loads to gravitational loads), with the following parameters considered:

λ	geometric scale	(3-4)
$\lambda_V = \sqrt{\lambda}$,	velocity scale	(3-5)
$(EA)_m = \lambda^3 (EA)_f$	axial rigidity	(3-6)
$(EI)_m = \lambda^5 (EI)_f$,	bending rigidity	(3-7)
$(GJ)_m = \lambda^5 (GJ)_f$,	torsional rigidity	(3-8)
$m_m = \lambda^2 m_f$,	mass per unit length	(3-9)
$mmi_m = \lambda^4 mmi_f$,	mass moment of inertia per unit length	(3-10)
$f_m = \lambda_V / \lambda \cdot f_f$	frequency	(3-11)

In the above-listed equations, *EA* denotes axial rigidity (*E* is modulus of elasticity and *A* is cross sectional area), *EI* (*I* is moment of inertia) denotes bending rigidity, *GJ* (*G* is shear modulus and *J* is torsion constant) is the torsional rigidity, the subscript *m* represents a model scale value while the subscript *f* represents a full-scale value. The above exponents are arrived at through dimensional analysis while keeping the Froude number constant between model-scale and full-scale. Note that while the model-scale speeds in the wind tunnel are low for typical aeroelastic studies of bridges, the instrumentation selected by RWDI has the sensitivity needed to resolve the wind-induced responses experienced by the model. Typical responses to be measured will include lateral, vertical and torsional deflections of the bridge deck, longitudinal and lateral accelerations at the maximum tower elevation, base moments of the towers and reference wind speeds in the wind tunnel.

During the model design, a finite element model of the structure will be developed at model scale to verify that the required simplifications to the aeroelastic model will yield the intended dynamic performance. The mode shapes and frequencies of the model will be compared to those provided by the design team for the full-scale bridge to ensure dynamic similitude. After model construction, the mode shapes and frequencies will be quantified to ensure they match the intended targets. In addition, the structural damping ratio in each mode of vibration will be quantified and adjusted accordingly to match the appropriate full-scale targets.

The aerodynamic stability of the bridge will be assessed using a low turbulence wind simulation. The wind-induced responses will be measured over a range of wind speeds and directions. At each wind direction and before any data sampling, a visual and real-time response inspection using on-line data analysis tools will be performed by the wind tunnel operator and technical coordinator to identify any potential for vortex-induced oscillation, galloping or flutter of the bridge. Once steady-state response is observed, time series records of data will be recorded. A similar procedure will be used for the measurement of buffeting responses with the turbulence simulation in the wind tunnel designed to match the site targets.

The buffeting response of the bridge will be assessed in a turbulent wind profile matching the site conditions as determined in RWDI's meteorological study. Measured responses of the structure will be used in conjunction with RWDI's buffeting response analysis to act as a calibration and to refine the final design wind loads for the completed structure and the investigated construction stages.

3.5 Project Flowchart

The manner in which the various activities of the wind tunnel test program impact final outcomes of the overall wind consulting services is described in the flow chart below.

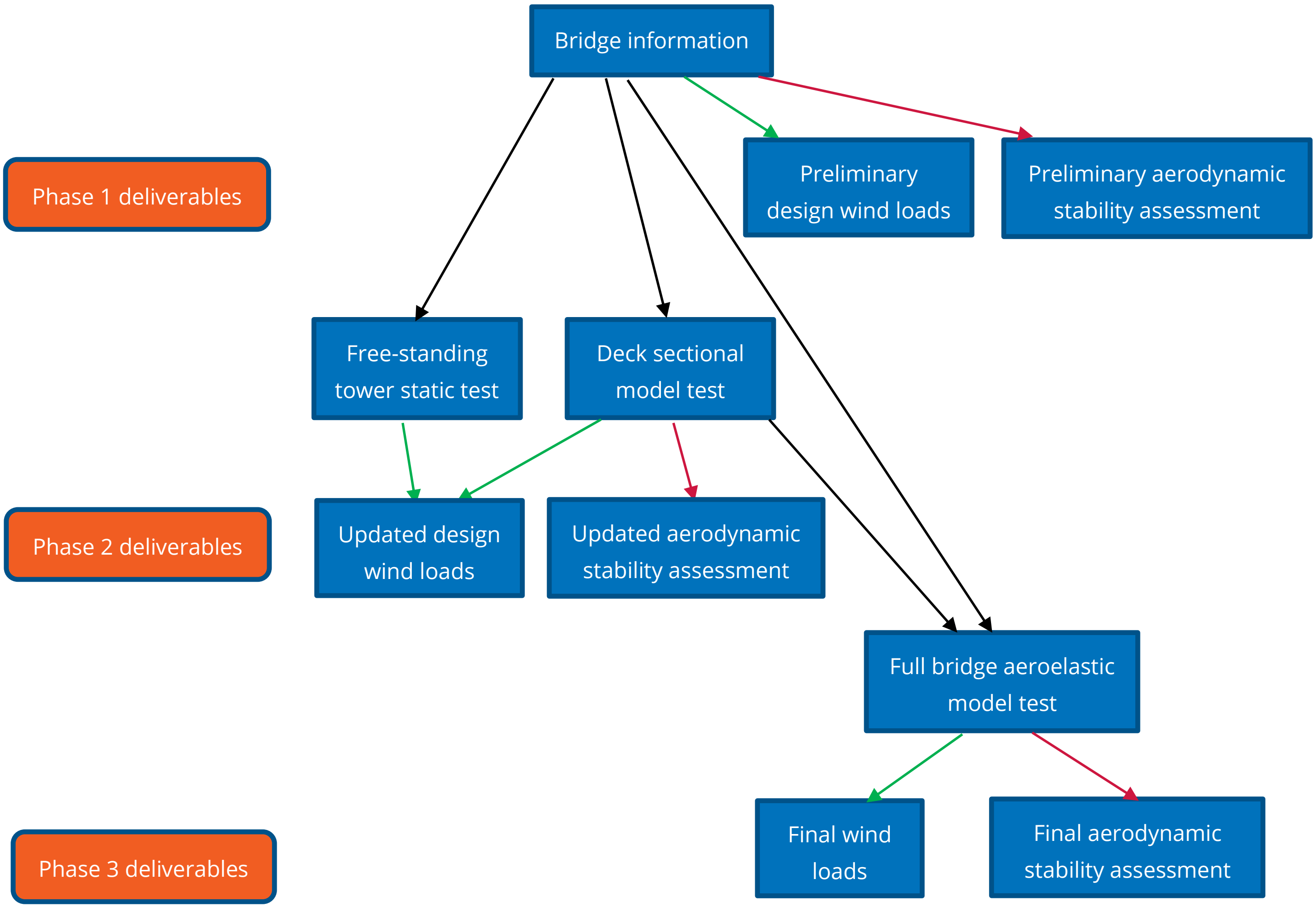


Image 3-1: Flow chart describing RWDI's wind consulting services and key deliverable stages.

4 DESIGN METHODOLOGIES

4.1.1 Design codes

The wind loading analysis for the Main Span Bridge, including site-specific climatology and wind tunnel testing, will be conducted according to ASCE/SEI 7 with a risk category of III.

4.1.2 Wind design speeds and pressures

See Section 2.3.2.

4.1.3 Evaluation of wind-induced accelerations

Wind-induced accelerations will be evaluated according to Article 3.8.3.3 of the AASHTO LRFD Bridge Design Specifications. This article states that peak accelerations of the superstructure shall be less than 5% of the acceleration due to gravity for steady wind speeds up to 30 mph and less than 10% of the acceleration due to gravity for steady wind speeds between 30 mph and 50 mph are the typical limits for pedestrian comfort.

4.1.4 Wind load combinations

The total wind loads for structural design are the peak loads, which include the mean wind loads, the background fluctuating wind loads, and the inertial loads due to the structural motions. The wind-structural response creates inertial loading particularly in the modes of vibration with the lowest frequencies where the wind contains the most significant turbulent energy. To estimate the overall load effects on the structure (such as stress or strain on each structural member), a general approach is to calculate the load effects for each load component and then use an appropriate statistical approach (such as the square-root-sum-of-squares method) to combine the peak dynamic effects due to the fluctuating loads and the inertial loads. However, this approach does not always fit the normal procedures of design offices. In view of this, sets of simplified wind load cases will be provided based on linear combinations of the dynamic loads in the various modes of vibration.

In each of the wind load combinations, the load patterns on the structure will be given as distributed vertical, lateral, along-the-bridge, and torsional moment loads, which must be applied simultaneously. Each of the load cases will present an individual maximized component in terms of the loading on the structure with various combinations of the modes of vibration. It is recommended that all provided load cases be considered, and that each main structural member should be designed based on the corresponding load case that gives the maximum load effects (i.e., stress and strain).

4.1.5 Aeroelastic phenomena

4.1.5.1 Flutter

Flutter is a self-excited aerodynamic instability that can grow to large amplitudes either in torsional motion alone or coupled torsional and vertical motions. All bridge decks will experience some form of flutter at a high enough wind speed. Since such an occurrence would likely result in a catastrophic collapse, the design criteria for flutter ensures that it only occurs at a wind speed that is associated with a high return period event.

4.1.5.2 Galloping

Galloping is a quasi-static type of instability whose occurrence is due to a negative rate of change in lift versus angle of attack. The negative rate of change in lift implies that the lift force is in-phase with the deck velocity which decreases the overall damping in the system as the wind speed increases. Once the overall damping in the system is negative the section may start to move vertically across the flow with an amplitude that grows rapidly with further increases in the wind speed.

4.1.5.3 Vortex induced oscillations (VIO)

Unlike the divergent behavior of galloping and flutter, there is another type of instability that is observed to be self-limiting in terms of its response to increasing wind speed. This type of instability is referred to as vortex-induced oscillation because it is caused by the alternate and regular shedding of vortices from both sides of a bluff body, such as a bridge deck or a cable. These types of vibrations can be tolerated provided their amplitudes, and associated accelerations, are not excessive or provided that the vibrations do not occur frequently. Typically for VIO, the main concerns are (i) serviceability; (ii) strength; and, (iii) fatigue. The serviceability concern is that excessive vibrations may be uncomfortable and/or visually disturbing to the users of the bridge. For this reason, the criteria for vortex shedding excitations are expressed in terms of maximum allowable accelerations.

4.1.5.4 Rain/wind induced vibration

Rain/wind induced vibration (RWIV) is a cable specific aerodynamic instability that can reach much larger amplitudes than amplitudes caused by vortex-shedding excitation. These vibrations are due to the aerodynamic effects caused when rivulets of water are running down cable surfaces. In the past, this effect has been observed to cause problems for bridge cables and has necessitated the development of a number of solutions. Disrupting the rivulets by adding helical fillets to the cable or its sheathing was found as the most effective way to reduce rain/wind vibrations. However, if the damping is sufficiently low, rain/wind oscillations can still occur.

4.1.6 Computer models of the bridge

A finite element model of the bridge will be developed by the design team and shared with RWDI for the creation of a simplified numerical model that will be used for the final stability analysis of the structure and the wind loading analysis. RWDI's simplified model will be essentially a strip-model of the bridge where appropriate masses, mass moments of inertia and aerodynamic properties of the sections can be defined. The model will be used in conjunction with RWDI's in-house buffeting response analysis software for the generation of windstorms matching the predicted site properties and for the calculation of the response of the structure. The equivalent static wind load cases will be provided in a format that is consistent with the design team's computer model of the bridge.

5 STATEMENT OF LIMITATIONS

The findings and conclusions presented in this report have been prepared for Kiewit Engineering Group Inc. (“Client”) and are specific to the project described herein (“Project”). The conclusions and recommendations contained in this report are based on the information available to RWDI when this report was prepared. Because the contents of this report may not reflect the final design of the Project or subsequent changes made after the date of this report, RWDI recommends that it be retained by Client during the final stages of the project to verify that the results and recommendations provided in this report have been correctly interpreted in the final design of the Project where appropriate.

The conclusions and recommendations contained in this report have also been made for the specific purpose(s) set out herein. Should the Client or any other third party utilize the report and/or implement the conclusions and recommendations contained therein for any other purpose or project without the involvement of RWDI, the Client or such third party assumes any and all risk of any and all consequences arising from such use and RWDI accepts no responsibility for any liability, loss, or damage of any kind suffered by Client or any other third party arising therefrom.

Finally, it is imperative that the Client and/or any party relying on the conclusions and recommendations in this report carefully review the stated assumptions contained herein and to understand the different factors which may impact the conclusions and recommendations provided.

6 REFERENCES

1 Irwin, P. Full aeroelastic model tests, Aerodynamics of Large Bridges, ed. A. Larsen, Balkema, Rotterdam, pp. 125-135, ISBN 90 5410 0427, 1992.

2 Tanaka, H. Similitude and modelling in bridge aerodynamics, Aerodynamics of Large Bridges, 1992 Balkema, Rotterdam pp 83-94.

TABLES

Table 2-1a: Recommended wind speeds at bridge site for winds from 140°

Wind Speed Applicable for	Return Period (years)	Mean Wind Speed (mph) at Deck Level 230 ft and Averaging Time		Corresponding 3-second Gust Speed (mph) at 33 ft Open Terrain
Design during construction	97	110.3	1 h	122.3
Design of completed bridge	1700	145.3	1 h	161.0
Stability during construction	1,000	129.4*	10 min	139.8*
Stability of completed bridge	10,000	150.1*	10 min	162.2*

*Includes reduction due to extreme wind climate directionality

Table 2-1b: Recommended wind speeds at bridge site for winds from 320°

Wind Speed Applicable for	Return Period (years)	Mean Wind Speed (mph) at Deck Level 230 ft and Averaging Time		Corresponding 3-second Gust Speed (mph) at 33 ft Open Terrain
Design during construction	97	98.2	1 h	122.3
Design of completed bridge	1700	129.3	1 h	161.0
Stability during construction	1,000	115.1*	10 min	139.8*
Stability of completed bridge	10,000	133.5*	10 min	162.2*

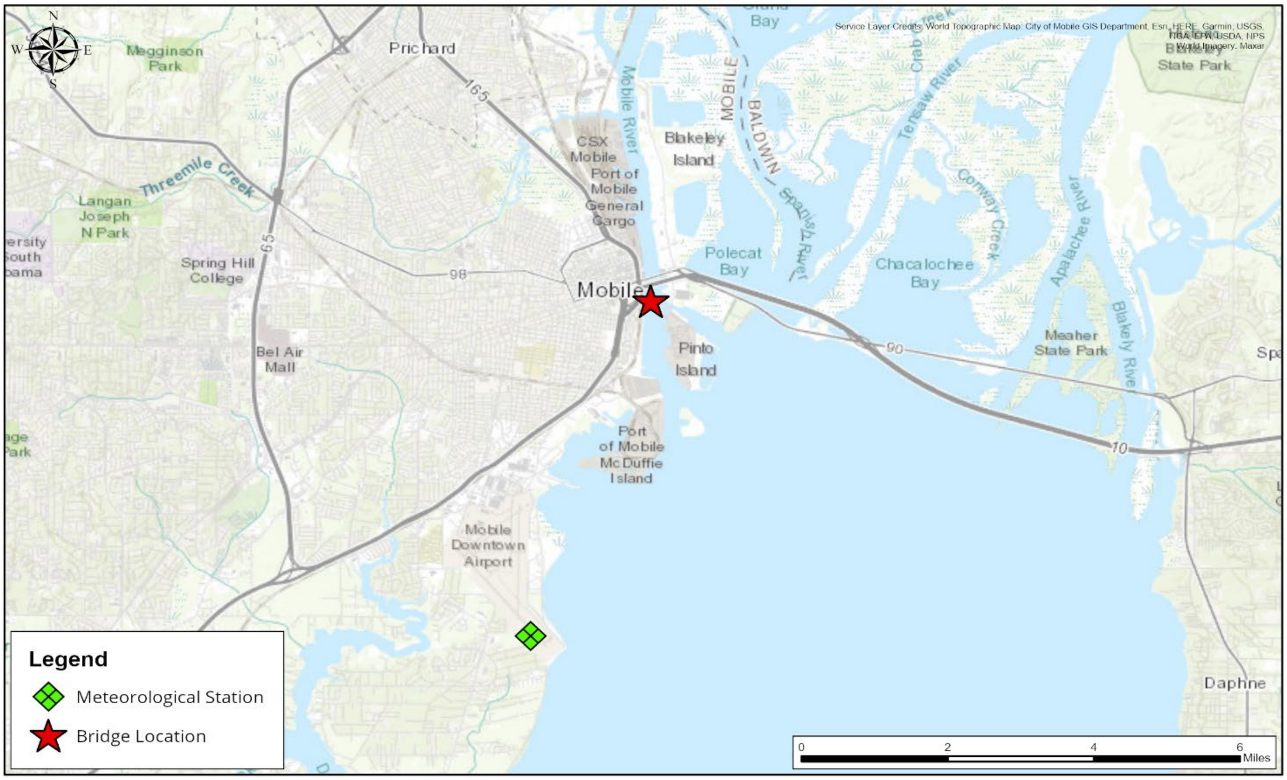
*Includes reduction due to extreme wind climate directionality

Table 2-2: Turbulence properties at deck level of 230 ft

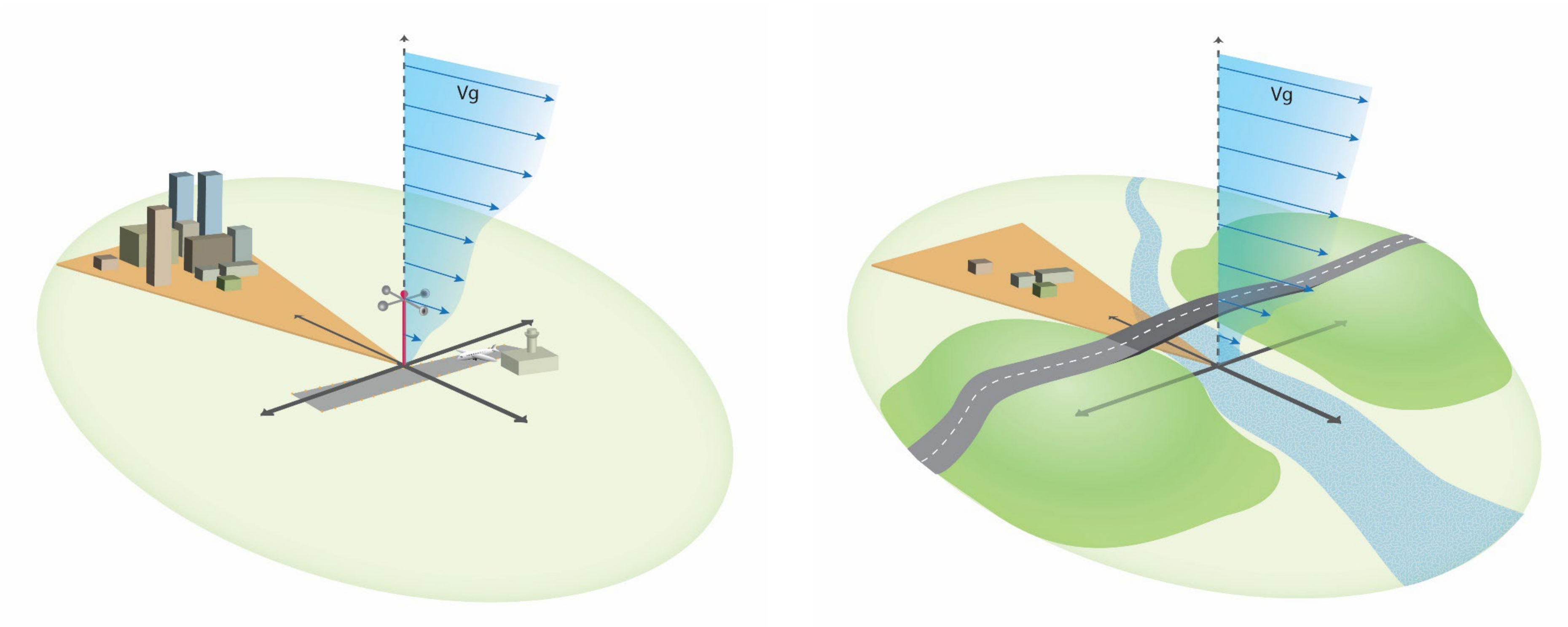
Direction (°CW from N)	Z_0 (ft)	α	I_u (%)	I_v (%)	I_w (%)	^xL_u (ft)	^xL_w (ft)	^yL_u (ft)	^yL_w (ft)	^zL_u (ft)
140	0.028	0.118	12.4	9.7	6.8	2573	214	701	117	425
320	0.869	0.173	18.9	14.7	10.4	2019	168	548	92	330

Notes:
1. z_0 - aerodynamic roughness
2. α - power law constant of wind profile
3. $I_{u,v,w}$ - longitudinal, horizontal-across-wind, and vertical turbulence intensities
4. $^{x,y,z}L_{u,v,w}$ - turbulence length scales

FIGURES



Location of the I 10 Mobile River Bridge and Climate Station	Figure: 2-1
I 10 Mobile River Bridge – Mobile, AB Project #2302137	Date: May 5, 2023

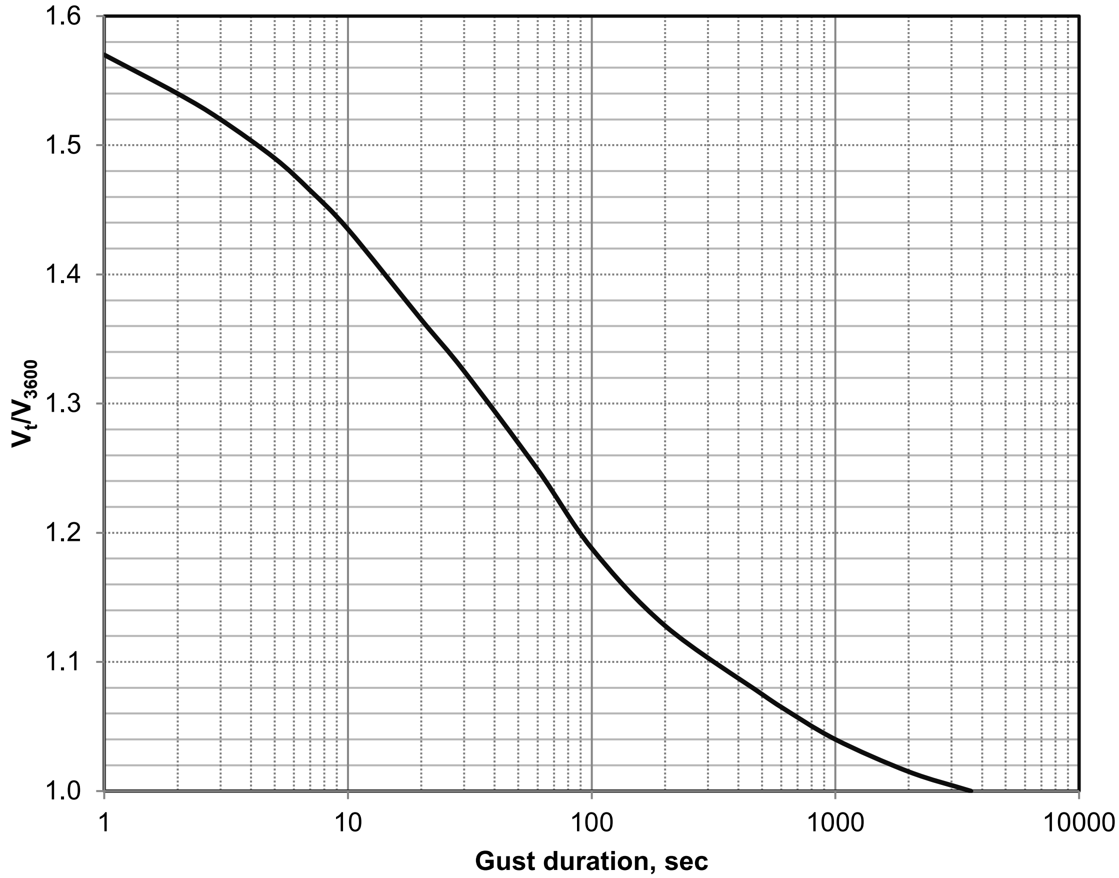


(a) Wind Profile at the Wind Measurement Location

(b) Wind Profile at the Bridge Site

The upwind terrain at the airport or wind measurement site (a) influences the wind speed profile differently than at the bridge site (b), up to gradient height, which is the height beyond which the surface roughness has any influence on the wind speed or turbulence. The ESDU method described in Section 2.2 of this report calculates the wind speed profile based on the changes in the upwind terrain and their relative distance to the measurement location, up to gradient height. The gradient height wind speed can then be similarly scaled down to the bridge deck height based on the upwind terrain at the bridge site. Note that these figures are meant to be illustrative in nature and not representative of the specific project site.

Translating Wind Speeds from Measurement Location to Project Site	Figure No. 2-2	
I 10 Mobile River Bridge – Mobile, AB Project #2302137	Date: May 5, 2023	



Notes:
1. From Figure C26-5.1 of ASCE 7-10
2. Gust to mean hourly speed ratios are only valid for open terrain

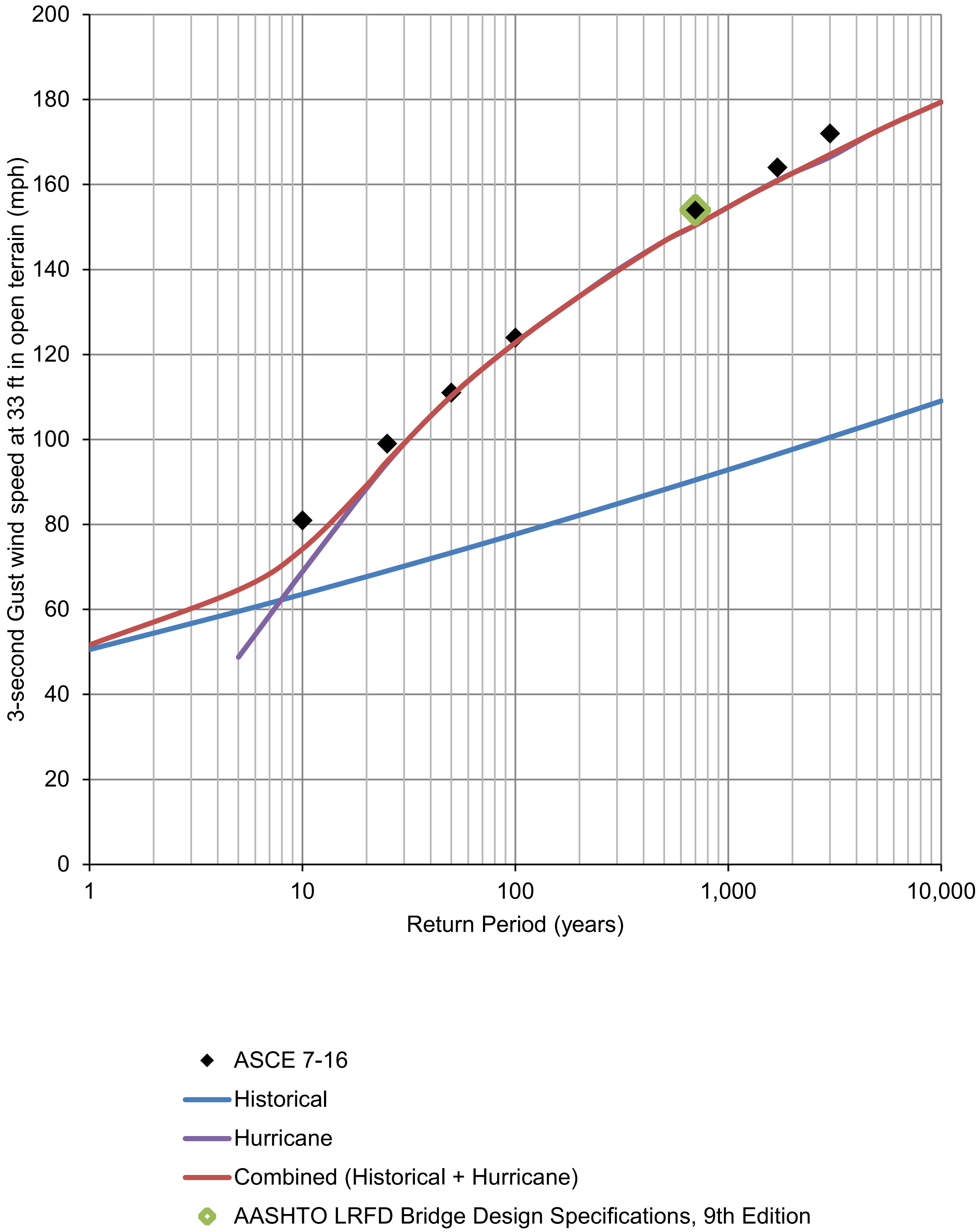
Gust to Mean Speed Ratio versus Wind Speed Averaging Time

I 10 Mobile River Bridge – Mobile, AB

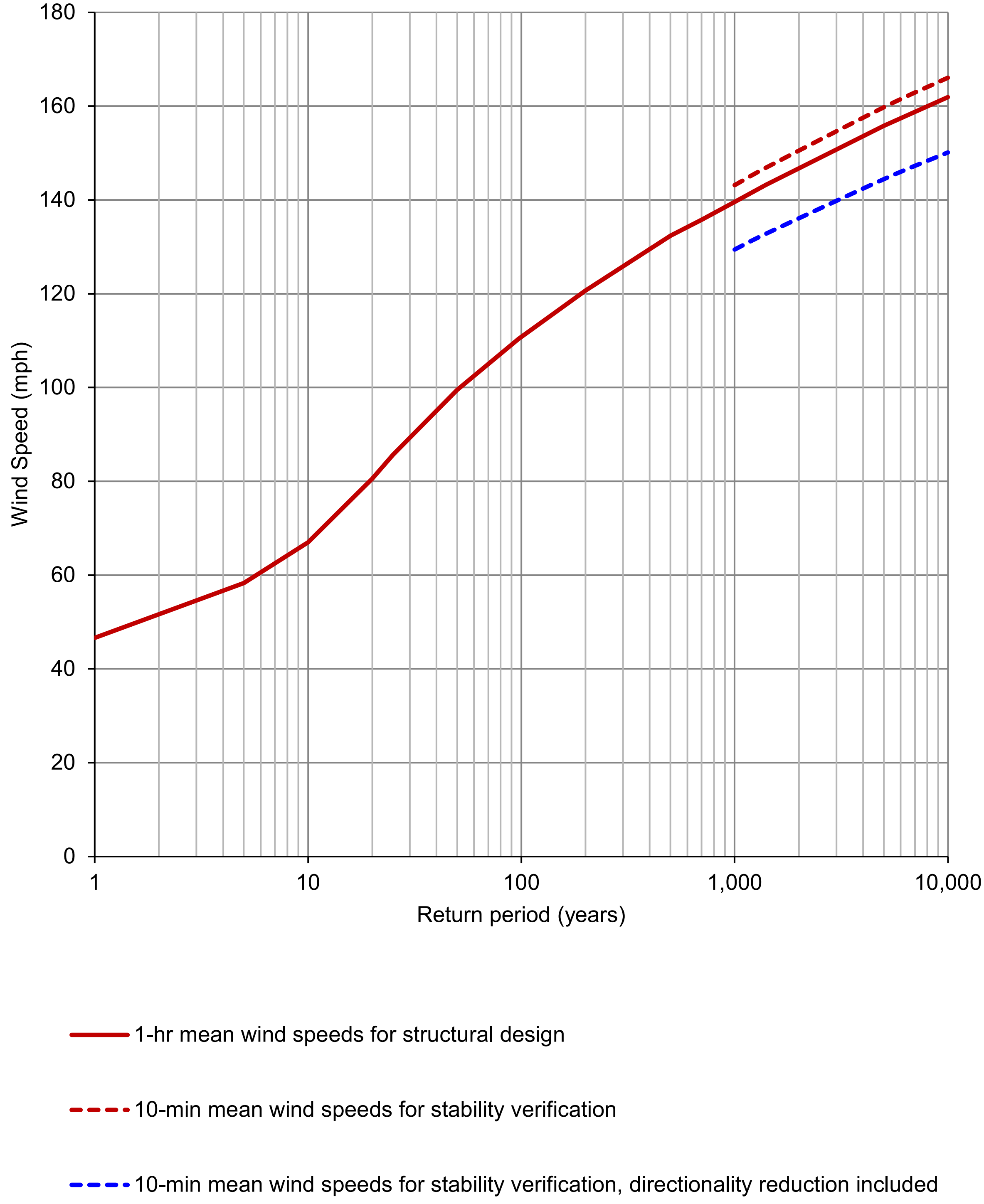
Project #2302137

Figure No. 2-3

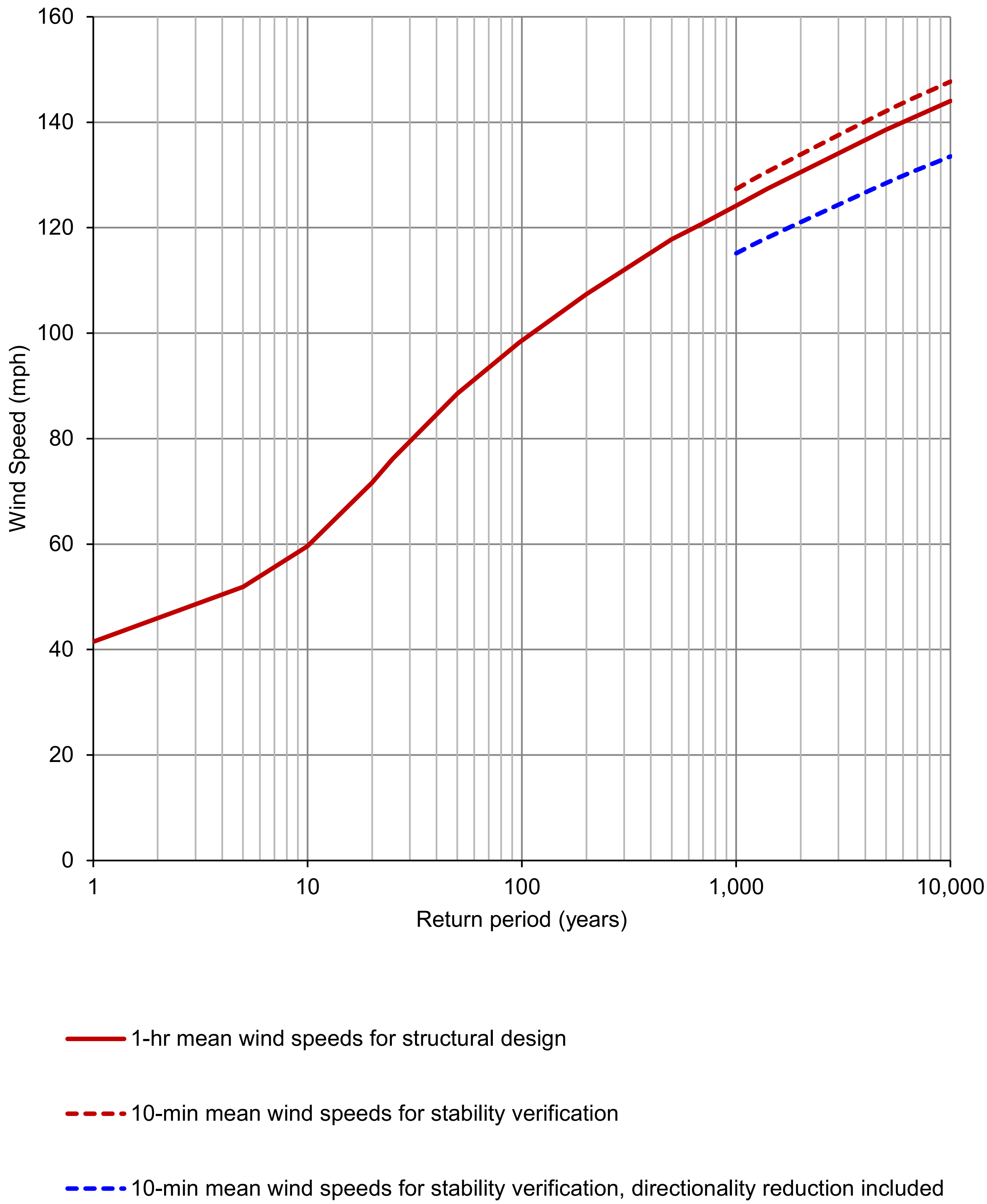
Date: May 5, 2023



3-second Gust wind speed at 33 ft in open terrain (mph)	Figure: 2-4
I 10 Mobile River Bridge – Mobile, AB — Project #2302137	Date: May 5, 2023



Mean Wind Speed for Various Return Periods, from 140º Wind speeds at elevation 230 ft above grade	Figure: 2-5a
I 10 Mobile River Bridge – Mobile, AB Project #2302137	Date: May 5, 2023



Mean Wind Speed for Various Return Periods, from 320º

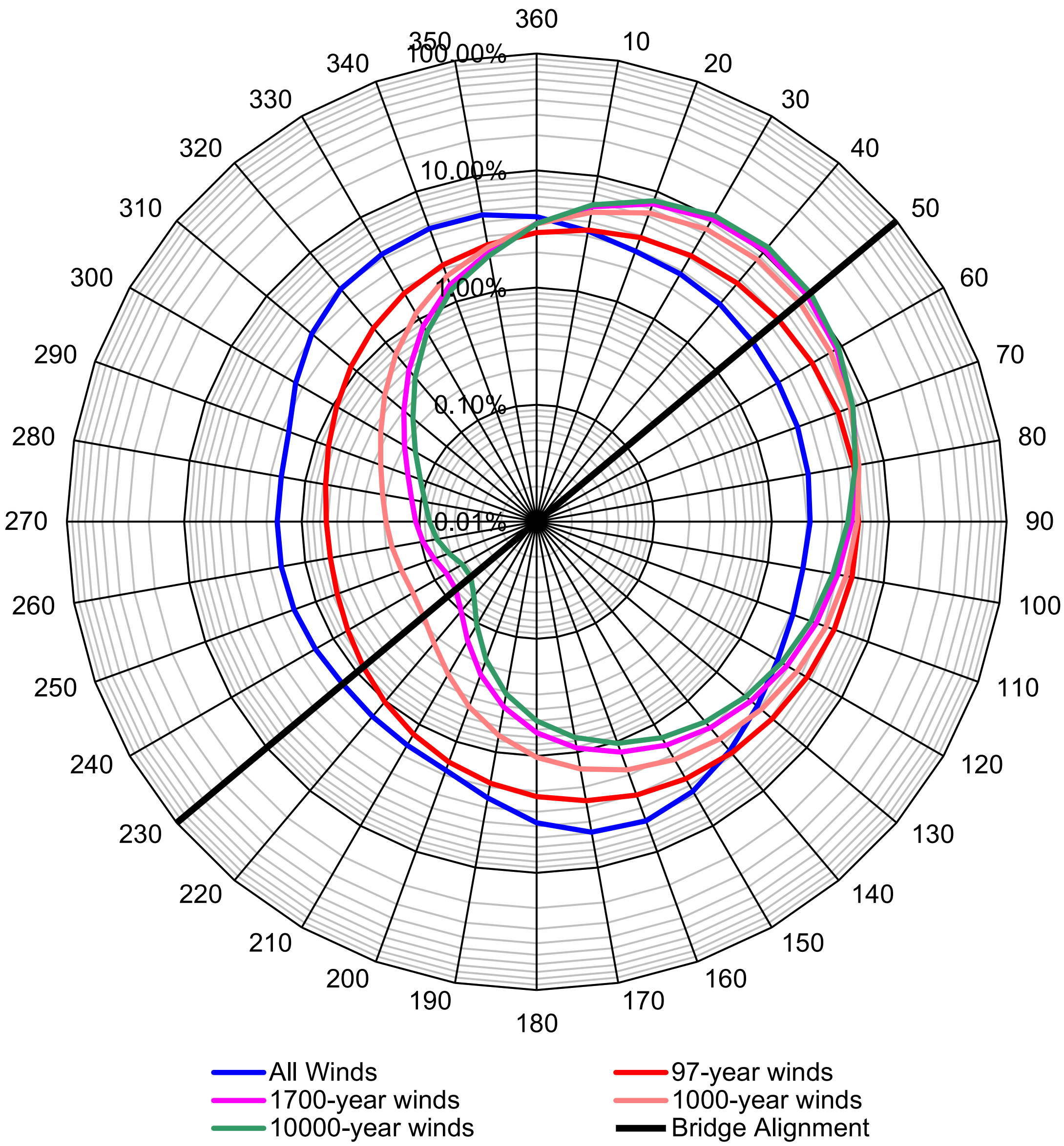
Wind speeds at elevation 230 ft above grade

I 10 Mobile River Bridge – Mobile, AB

Project #2302137

Figure: 2-5b

Date: May 5, 2023



Directional Distribution of Hourly Mean Winds at the Project Site	Figure: 2-6
I 10 Mobile River Bridge – Mobile, AB Project #2302137	Date: May 5, 2023

APPENDIX A

APPENDIX A
WIND CLIMATE ANALYSIS PROCEDURES

A.1 TERRAIN CORRECTIONS

Special attention is given to the analysis of the hourly records to account for the effects of the terrain surrounding an anemometer. Typically, anemometers are installed in an open terrain exposure that is used as a reference condition by building codes. However, this is rarely the case in real world applications. This means the true exposure of the anemometer is not that of the standard open terrain conditions. It is important to take this impact into account so as to avoid underestimating or overestimating design wind speeds.

Prior to conducting any analysis using the surface observations, the effect of upwind terrain roughness and land cover characteristics on the wind speeds at the anemometer station is assessed for each wind direction, and used to adjust wind speeds to a standard open terrain profile.

ESDU[1],[2] describes a method based on the work of Deaves and Harris[3] for evaluating changes in the mean velocity profile following a change in ground roughness. This is particularly useful when analyzing meteorological records from an anemometer surrounded by varying terrain roughness for different wind directions.

This method is used to determine anemometer exposure. Maps, photographs and satellite imagery of the location are used to assess the ground roughness changes for each wind direction. The wind speeds for each wind direction were then adjusted based on the exposure of the anemometer to produce wind speeds that are equivalent to standard open terrain.

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1 ESDU (1982) *Strong Winds in the Atmospheric Boundary Layer. Part 1: Mean Hourly Speeds*, Item 82026, Issued September 1982 with Amendments A and B April 1993. Engineering Sciences Data Unit, ESDU International, 27 Corsham Street, London N16UA.

2 ESDU (1983) *Strong Winds in the Atmospheric Boundary Layer. Part 2: Discrete Gust Speeds*, Item 83045, Issued November 1983 with Amendments to 1993. Engineering Sciences Data Unit, ESDU International, 27 Corsham Street, London N16UA.

3 Deaves, D.M. and Harris, R.I. (1978) *A Mathematical Model of the Structure of Strong Winds*, Construction Industry Research and Information Association (U.K.), Report #76.

A.2 Prediction of Extreme Wind Speeds and the Method of Independent Storms

The first step in conducting an extreme value analysis for predicting wind speed frequency is producing a set of independent maxima which will ultimately be fitted to an extreme value distribution. Traditionally, this would be done using readily available data sets of annual maxima. Since the resolution of this data is relatively low, the likelihood of neighboring years having maxima that are related to the same wind event is quite low, and so they can generally be assumed to be independent. However, using annual maxima for this purpose means that many high wind events that occur will not be considered in the assessment of risk if they are not the highest event in a given year. To illustrate this, consider the 2 years of time series of wind data in Figure 1. There are actually three independent wind events of higher speed in the year 1964 than in the year 1965. Considering only the annual maxima would result in 2 of those high wind events being ignored, and could result in an under-prediction of the true risk.

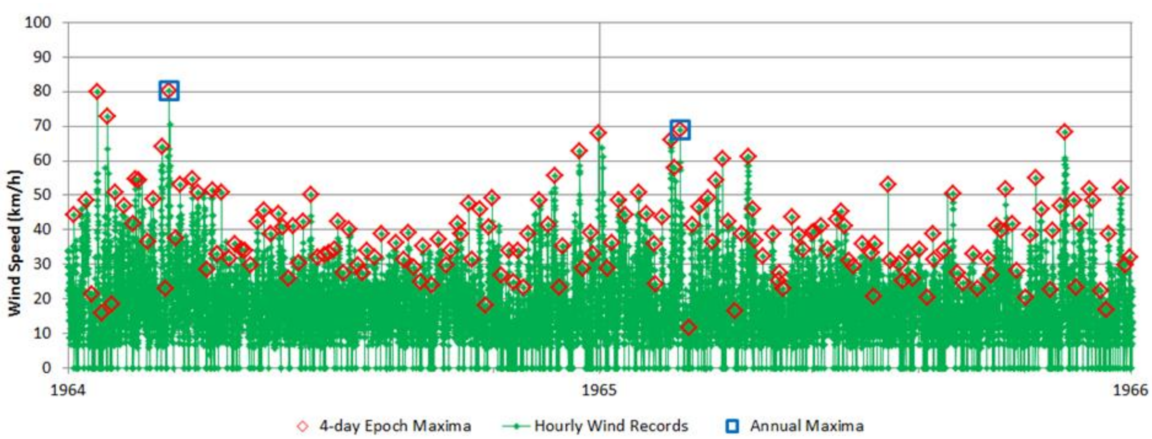

Figure 1: 4-day Epoch Maxima in Comparison to Annual Maxima

As you reduce the size of the epoch considered for selecting maxima, you also increase the probability of selecting maxima from neighboring epochs that are actually part of the same wind event, so it becomes increasingly important to verify independence. The Method of Independent Storms (MIS) is an extreme value technique described by Cook[4], and then subsequently updated by Harris[5]. As the name suggests, MIS ensures that the maxima included for extreme value fitting are selected from independent events.

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[4] Cook, N.J. (1982) *Towards better estimation of extreme winds*, Journal of Wind Engineering and Industrial Aerodynamics 9, pp.295-323.

[5] Harris, R.I. (1999) *Improvements to the method of independent storms*, Journal of Wind Engineering and Industrial Aerodynamics 80, pp.1-30.

RWDI's implementation of MIS separates the historical dataset into 4-day epochs. The selection of a 4-day epoch is based on the wind power spectrum, which tends to peak at approximately 4 days due to the normal duration of a synoptic pressure system in an extra-tropical climate. Within the 4-day epoch, a local minimum is determined. This forms a series of minima throughout the historical dataset. A second set of epochs is defined by the minima, between which a local maximum is selected. This process is illustrated in Figure 2.

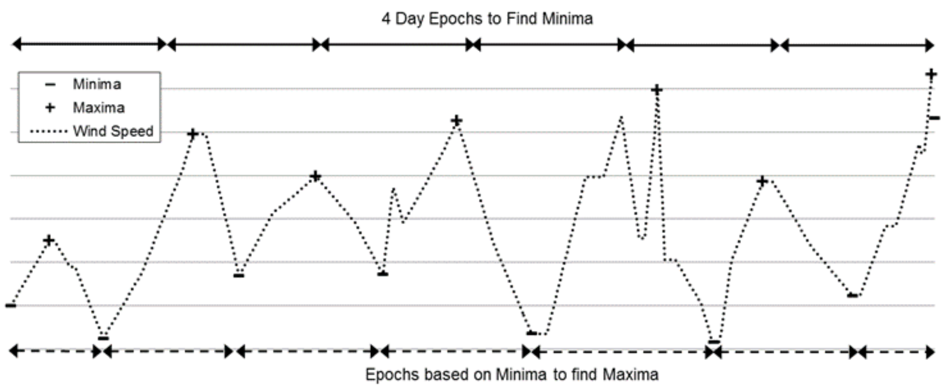

Figure 2: Maxima and Minima-Finding Routine

This process is applied to the entire dataset and the maxima are ranked according to wind speed. Finally, each wind speed is assigned a probability based on rank according to the Gringorten Probability Method[6]. These speeds are fit to a Fisher-Tippet Type I distribution, which is given by:

$$P(\hat{U}) = e^{-e^{-y}}, \qquad (1)$$

where $P(\hat{U})$ is the probability that the annual peak velocity will not exceed the value, and $\hat{U}$ is the peak velocity.

The parameter y is defined as:

$$y = a(\hat{U} - b), \qquad (2)$$

in which $1/a$ is dispersion and b is the mode.

[6] Gringorten, I.I. (1963) A plotting rule for extreme probability paper, Journal of Geophysical Research 68(3), pp.813-814.

A.3 Directional Analysis of Wind Speeds

A commonly used mathematical expression for wind statistics is the Weibull expression, which states:

$$P_\theta(U) = A_\theta e^{-\left(\frac{U}{C_\theta}\right)^{K_\theta}}, \tag{3}$$

$P_\theta(U)$ being the probability that the mean wind speed will exceed the value U when its wind direction is within the azimuthal sector θ. The A_θ factor is the fraction of the time that the wind blows from the selected sector, and C_θ and K_θ are the velocity and shape parameters required for each of the direction sectors. A variant on this Weibull expression that fits the higher wind speeds separately, and ultimately provides a better fit to the higher wind speeds, is also applied. The expression for this fit is:

$$P_\theta(Uz) = \begin{cases} \text{if } Uz < U_{th_\theta}, & A_\theta e^{-\left(\frac{Uz}{C_{L\theta}}\right)^{K_{L\theta}}} \\ \text{if } Uz \geq U_{th_\theta}, & A_\theta e^{-\left(\frac{Uz}{C_{U\theta}}\right)^{K_{U\theta}}} \end{cases} \tag{4}$$

where $C_{L\theta}$ and $K_{L\theta}$ are the Weibull parameters for wind speeds below the threshold velocity U_{th_θ}, and $C_{U\theta}$ and $K_{U\theta}$ are the Weibull parameters for wind speeds greater than or equal to the threshold velocity. The Weibull expression was fitted to the data from the meteorological station using wind direction intervals of 10 degrees.

To compute the functional relationship between wind speed and return period the upcrossing method described by Lepage and Irwin[7] (1985) and Irwin[8] (1988) was applied where it can be shown that the frequency F of an event where the wind velocity Uz at a height z will be exceeded is given by:

$$F(> Uz) = \frac{1}{2}\sum_\theta \left|\overline{\dot{U}z}\right| \frac{dP_\theta(U_z)}{dU_z}, \tag{5}$$

from which the return period R of wind velocity Uz may be determined as:

$$R = \frac{1}{n\,F(>Uz)}, \tag{6}$$

where n is the number of hours per year and $\left|\overline{\dot{U}z}\right|$ is the mean absolute rate of change of the hourly wind velocities. Irwin (1988) derived the following empirically based equation which applies:

$$\left|\dot{U}z\right| = 0.065Uz + 1.8e^{-0.07Uz}, \tag{7}$$

where $\left|\dot{U}z\right|$ has the units of km/h/h.

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7 Lepage, M.F. and Irwin, P.A. (1985) A technique for combining historical wind data with wind tunnel tests to predict extreme wind loads. Proceedings of the 5th National Conference on Wind Engineering, Lubbock, Texas, Session 2B-71, Nov. 6-8.

8 Irwin, P.A. (1988) Pressure model techniques for cladding loads. Journal of Wind Engineering and Industrial Aerodynamics, 29, pp.69-78.