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16. Abstract

The purpose of this study is to assess the sensitivity of four overpass bridges to wind and traffic loadings. The bridges are planned for construction on the U.S. 59 in Houston, Texas. Each bridge will be of the tied-arch type and will be constructed on-grade over cut areas with clear spans of 228 ft (69.5m). To expedite construction, unique procedures are planned which use existing bridges as platforms for final fabrication of the tied arches. This procedure requires consideration of wind loadings on the arches during construction.

This report provides an overview of the wind effects on bridges in the Houston area. The results of analytical and laboratory (wind tunnel) studies of the planned tied-arch bridges are published in a report (Research Study No. 7-1982-2, December 1994): the results are not repeated here. The overall conclusion of the analytical and laboratory studies was that the proposed arch bridges over U.S. 59 in Houston can resist anticipated wind and traffic loadings when completed. The research task of field measurements during and after construction, which was part of the proposed study, is postponed because of extended delay in initiation of construction. The wind climate in the Houston area and possible vulnerability of arches to lateral wind loads during construction suggest the desirability of field measurements of the response of these bridges during and after construction. The field measurements provide validation of analytical and laboratory procedures which, then, can be used in the future with confidence.

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SENSITIVITY OF OVERPASS BRIDGES TO WIND AND TRAFFIC LOADING

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Research Report Number 7-1982-3F

conducted for

Texas Department of Transportation

by the

DEPARTMENT OF CIVIL ENGINEERING

TEXAS TECH UNIVERSITY

January 1997

IMPLEMENTATION STATEMENT

This project will yield several products useful to the Texas Department of Transportation (TxDOT), including: expected wind loadings on four overpass bridges planned for the Houston District, confirmation of structural analyses conducted by TxDOT, and guidance for final bridge design and bridge erection.

FEDERAL/DEPARTMENT CREDIT

Prepared in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

AUTHOR'S DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view of policies of the Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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ENGINEERING DISCLAIMER

Not intended for construction, bidding, or permit purposes. The engineer in charge of the research study was Kishor C. Mehta, P.E., Texas 24735.

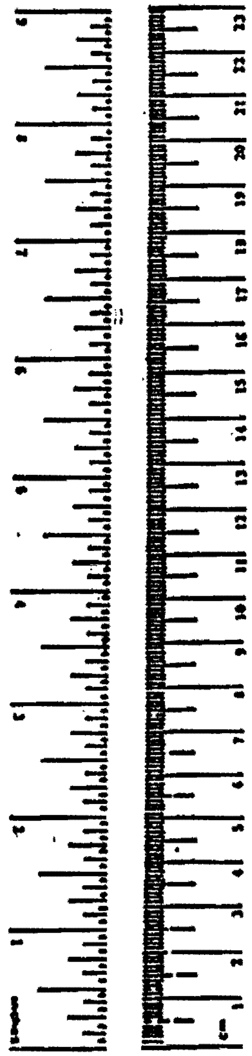
TRADE NAMES AND MANUFACTURERS' NAMES

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

METRIC CONVERSION FACTORS

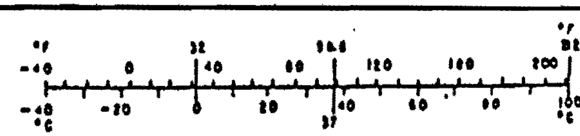
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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SUMMARY

The purpose of this study is to asses the sensitivity of four overpass bridges to wind and traffic loadings. The bridges are planned for construction on the U.S. 59 in Houston, Texas. Each bridge will be of the tied-arch type and will be constructed on-grade over cut areas with clear spans of 228 ft (69.5m). To expedite construction, unique procedures are planned which use existing bridges as platforms for final fabrication of the tied arches. This procedure requires consideration of wind loadings on the arches during construction.

This report provides an overview of the wind effects on bridges in the Houston area. The results of analytical and laboratory (wind tunnel) studies of the planned tied-arch bridges are published in a report (Research Study No. 7-1982-2, December 1994); the results are not repeated here. The overall conclusion of the analytical and laboratory studies was that the proposed arch bridges over U.S. 59 in Houston can resist anticipated wind and traffic loadings when completed. The research task of field measurements during and after construction, which was part of the proposed study, is postponed because of extended delay in initiation of construction. The wind climate in the Houston area and possible vulnerability of arches to lateral wind loads during construction suggest the desirability of field measurements of the response of these bridges during and after construction. The field measurements provide validation of analytical and laboratory procedures which, then, can be used in the future with confidence.

I. INTRODUCTION

1.1 Background

Four overpass bridges over U.S. 59 in the city of Houston, Texas are planned for replacement. The reason for replacing the overpass bridges is to provide additional highway lanes for U.S. 59. Upgrading of U.S. 59 mandates that intermediate piers of the overpass bridges and median on U.S. 59 be eliminated. The new bridges are required to span the full width of the U.S. 59 highway, which is 228 ft (69.5m).

Consideration of several factors including clearance, existing storm sewers, existing city street levels and construction space led to use of the tied-arch system to support overpass bridges. The vibrational characteristics of tied-arch bridges with moderate spans make them particularly sensitive to wind and live loadings. The Texas Department of Transportation (TxDOT) commissioned Texas Tech University to assess effects of traffic and wind loads on tied-arch bridges with clear span of 228 ft (69.5m). If the assessment indicated strength or vibration problems with the new bridges, the researchers were to suggest mitigation measures. The overpass bridges may or may not have traffic message signs or other advertisement signs attached to them. A phase of construction when steel tied-arches are erected before the bridge deck is installed is also required to be checked for strength and stability.

1.2 Objective

The objective of this study is to assess the sensitivity of four overpass tied-arch bridges to wind and traffic loadings. The wind loading includes wind environment of hurricane storms that may come ashore in Galveston and pass over Houston.

The primary objective of the study is accomplished through analytical study and laboratory testing in a wind tunnel. A geometrically-scaled model of the bridge cross section is tested in a wind tunnel to determine aerodynamic parameters including that of vortex shedding. These aerodynamic parameters and traffic patterns are used in the available analytical procedures to asses the response of the bridge.

An additional objective of validating the laboratory testing and analytical procedure through field measurements during and after construction is postponed because of extended delay in initiation of construction. This objective can be pursued in the future when the construction of the four bridges is initiated. Accomplishment of this objective is important so validated analytical and laboratory procedures can be used in the future with confidence.

1.3 Conclusions of Analytical and Laboratory Studies

The results of analytical and laboratory (wind tunnel) studies of the tied-arch bridges are published in a report (Research Study No. 7-1982-2, December 1994); the results are not repeated here. The overall conclusion from the analytical and laboratory studies is that the proposed arch bridges over U.S. 59 in Houston can resist anticipated wind and traffic loadings. For this particular length of clear span and design of deck and arches, the bridge section is aeroelastically stable[1] (not susceptible to flutter instability) until the windspeed reaches above 350 mph (physically unrealizable). The so called "lock in" windspeed for vortex shedding, which can cause steady vibrations, is 81 mph.[2] The stiffnesses of these bridges are such that they are not likely to become unstable by flutter or are not likely to be excited by vortex shedding until the windspeeds are fairly high. Since these phenomena require persistent wind for long durations, the structures are not likely to become unstable.

A logical conclusion of the overall project is that the analytical and laboratory procedures used are appropriate for this tied-arch bridge; however, they need to be validated with field measurements for their future use. If a tied-arch bridge with the same deck parameters but with a clear span of 400 ft is to be constructed, it is likely to be unstable in flutter and vortex shedding phenomena according to the current analytical and laboratory procedures. If field measurements are pursued to validate the procedures, problems in future designs can be anticipated.

1.4 Content of the Report

this report contains a description of wind climate under various types of windstorms and a general discussion of bridge design for wind effects. The discussion includes analytical procedures and wind tunnel techniques, as well as the current standard of practice for wind effects. Recent experiences as well as potential benefits of field testing are outlined. Conclusions and recommendations are provided in a succinct manner.

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[1]The bridge decks without traffic signs are always stable. The ones with traffic signs became unstable beyond 350 mph.

[2]The steady vibrations do not occur if the critical damping of the bridge is above 1%.

2. DESIGN WINDSPEEDS

There are prevailing winds on a daily basis and occasional high winds in gust fronts, thunderstorms and other windstorms throughout the country. The interest to engineers is the extreme wind climate that can damage or destroy a building or other structure. It is not appropriate to design a structure using maximum recorded windspeed for a given site. The design windspeed values are established on a probabilistic basis by national standards and codes. A general discussion of design windspeed that affects the Houston area is presented in this section.

2.1 Design Winds

Extreme winds occur in windstorms such as thunderstorms, gust fronts, hurricanes and tornadoes. Even though the contiguous United States experiences close to 1,000 tornadoes annually, the probability of a tornado occurring at a given site is very small because of the short duration of the storm and the small area affected by a tornado. Hurricane storms spawn in the Atlantic and hurricane winds can affect the East and Gulf coastal areas of the country. Most of the country is effected by thunderstorms and gust front winds.

In order to assess wind climate on a probabilistic basis, it is necessary to have annual maximum windspeed at a site for a continuous number of years. The windspeed data are statistically reduced using extreme value analysis procedure based on Fisher-Tippett Type I (Gumbel) distribution (Simiu and Scanlan, 1986). This statistical procedure provides the windspeed associated with annual probability of exceeding or mean recurrence interval. The mean recurrence interval is the reciprocal of annual probability of exceeding windspeed.

It is essential that design professionals understand the mean recurrence interval in terms of probability of exceeding design windspeeds during the service life of a structure. The probability, P, that the design windspeed will be exceeded at least once during service life is given by the expression: $P=1-(1-P_a)^n$.

Where, P_a is the annual probability of exceeding (reciprocal of mean recurrence interval) and n is the service life of a structure. Thus, the probability that a structure will experience windspeeds equal to or exceeding design windspeed depends on the expected life of a structure and mean recurrence interval.

As indicated in Table 1, there is a 64% chance of exceeding design windspeed if the annual probability is 0.02 (50 year mean recurrence interval) and design life of the structure is 50 years. The probability of experiencing design windspeed reduces to 40% if the same structure is designed for a windspeed associated with an annual probability of 0.01 (100) year mean recurrence interval). It is a common practice in this country to use design windspeed associated with 50 year mean recurrence interval.

The load factors or allowable stresses reduced from yield strength provide needed safety factors.

2.2 Design Windspeed for Houston

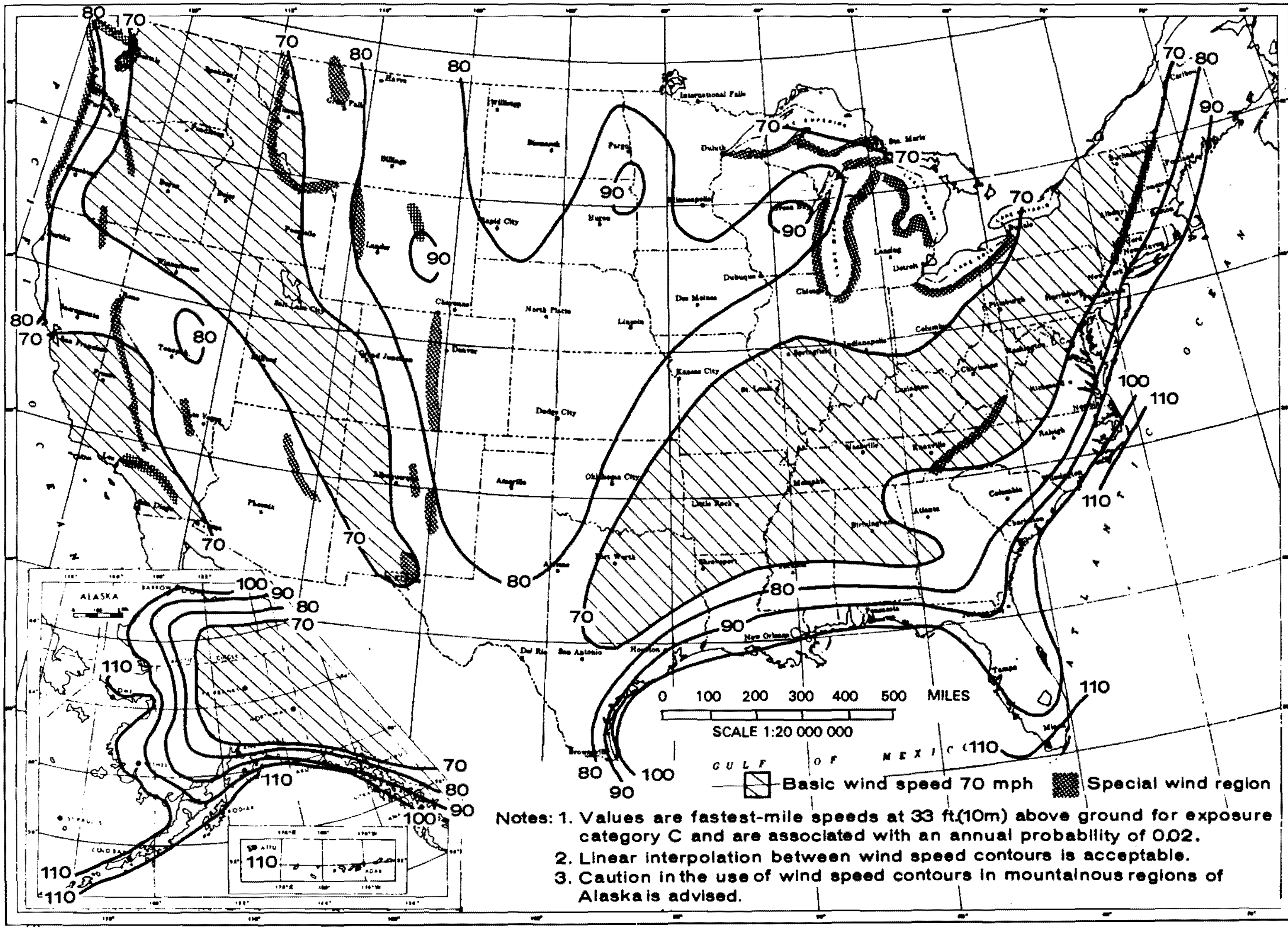
American Society of Civil Engineers Standards Committee 7 on minimum design loads develops a design windspeed map. A basic windspeed map shown in Figure 1 was originally developed for the American National Standards Institute standards ANSI A58.1-1982 (ANSI, 1982). The windspeeds in the map are fastest-mile speeds at 33 ft (10m) above ground for flat and open terrain (Exposure C) associate with an annual probability of 0.02 (mean recurrence interval of 50 years). The windspeeds shown in the map were established from data collected at the National Weather Stations throughout the country and subjecting the data to extreme value statistics. The application of statistics necessitates that data set include a number of windspeed values at a given site. along the hurricane-prone East Coast and Gulf Coast regions there were not sufficient number of hurricanes striking a given location to avail the data to statistical analysis. The windspeed contours in hurricane-prone coastal regions were established using Monte Carlo simulation of hurricane storms. The simulation technique provided sufficient data to establish windspeeds. The windspeed map produced for ANSI A58.1 standard was adopted in the consensus standard ASCE7-88 (ASCE, 1990). The design windspeed value for Houston area is 90 mph.

TABLE 1

PROBABILITY OF EXCEEDING DESIGN WINDSPEED

Annual	Mean Recurrence	Life of Structure, n				
Probability, P_a	Interval Years	1	10	25	50	100
0.04	25	0.04	0.34	0.64	0.87	0.98
0.02	50	0.02	0.18	0.40	0.64	0.87
0.01	100	0.01	0.10	0.22	0.40	0.64
0.002	500	0.002	0.02	0.05	0.10	0.18

Figure 1 Design Wind Speed for Buildings and Structures (ASCE 7-88)



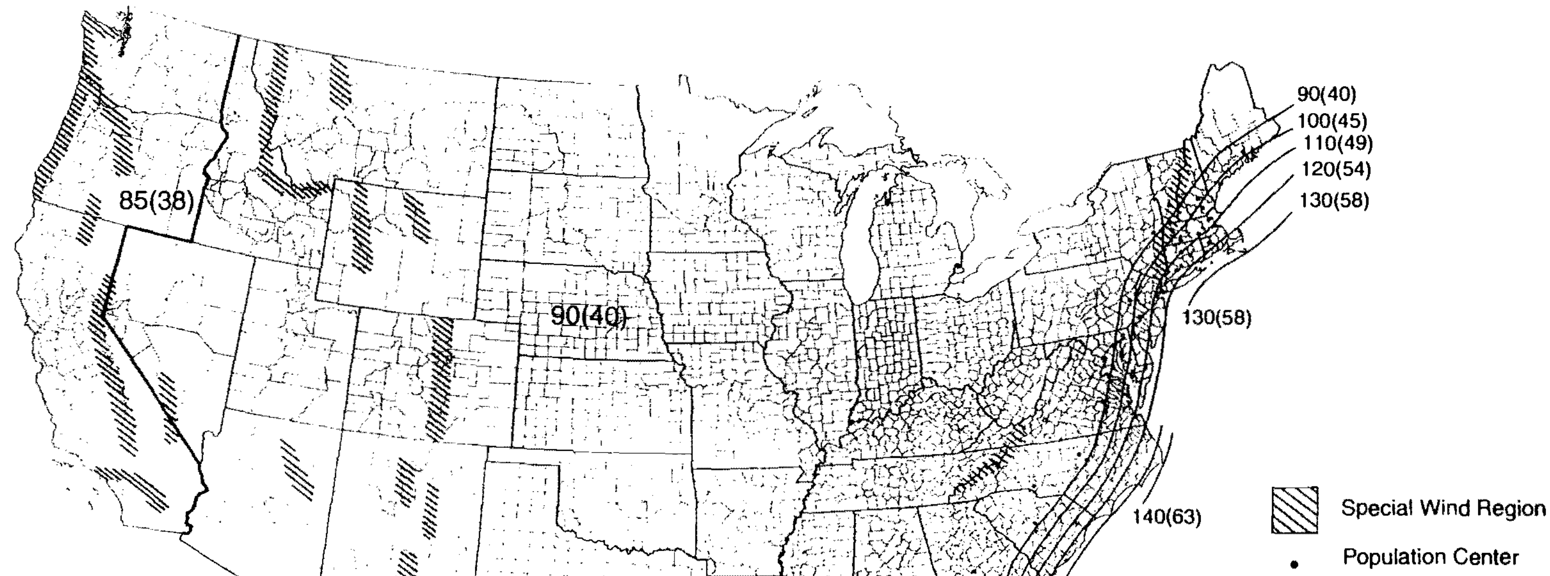
In the ASCE7-88 (ASCE, 1990), the hurricane region windspeeds are multiplied by an Importance Factor of 1.05. This Importance Factor accounts for difference in probability distribution of hurricane windspeeds and windspeeds at inland stations (Mehta et al, 1991). With this Importance Factor the design windspeed for the Houston area is close to 95 mph.

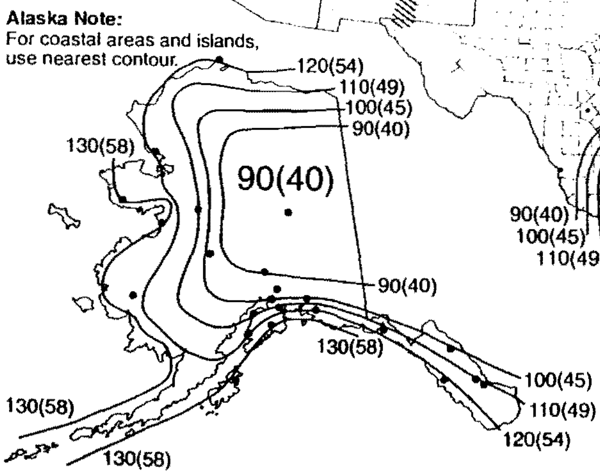
The ASCE7 standard on minimum design loads was revised in 1995 (ASCE, 1995). A significant change in the standard is the use of 3-second gust speed instead of the fastest-mile speed as the basic windspeed. The reason for the change is the discontinuation of collection and archival of fastest-mile windspeed data by the National Weather Service in mid-1980's. The hurricane-prone region windspeeds are still established by Monte Carlo simulation though recent work is incorporated into the contours of the new map (Krayer and Marshall, 1992; Georgiou et al, 1983; Peterka and Shahid, 1993; Vickery and Twisdale, 1993). The new windspeed map of ASCE7 standard is shown in Figure 2. These windspeeds are 3-second gust speeds at 33 ft (10m) above ground in Exposure C and associated with annual probability of 0.02 (50 year mean recurrence interval). In Figure 2, windspeed contours are provided only in hurricane-prone coastal regions; the rest of the country has uniform windspeed of 85 or 90 mph. The windspeed contours have incorporated Importance Factors for hurricanes. Basic 30-second windspeed for Houston in the map is 220-120 mph.

Even though the windspeed value of 110-120 mph is much higher than 95 mph, the resulting loads from the two standards are comparable because of adjustments in height and terrain factor and in gust effect factor.

Basic reference windspeed maps do not include localized windstorm events of downburst and tornadoes. These windstorms are short lived and cover limited area. Probability of these storms striking a given spot in Houston area is likely to be less than annual probability of 0.001 (1000 year mean recurrence interval). Even with this small probability, there is always a chance that a given structure can experience windspeeds significantly higher than design windspeeds.

Figure 2 Design Wind Speed for Buildings and Structures (ASCE 7-95)





Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	125	(56)
Guam	170	(76)
Virgin Islands	125	(56)
American Samoa	125	(56)

Notes:

1. Values are 3-second gust speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category and are associated with an annual porbability of 0.02.
2. Linear interpolation between wind speed contours is permitted.
3. Islands and coastal areas shall use wind speed contour of coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special regions shall be examined for unusual wind conditions.

3. DESIGN FOR WIND EFFECTS

3.1 Introduction

Prior to the dramatic failure of the Tacoma Narrows bridge in 1941 several flexible bridges had suffered disturbance and damage by the wind. However, it was the Tacoma Narrows bridge disaster that paved the way for serious considerations of wind-induced loads in the design of flexible bridges. TxDOT like other Departments of Transportation has been involved in designing and constructing different types of bridges of various span lengths. Some of these bridges are flexible or semi-flexible. It is this category of bridges – with medium to long spans, suspended by cables or cable-stays or arch and with aerodynamically blunt decks – which are vulnerable to the brute force of wind and requires special attention. This section of the report will attempt to explain the various effects of wind on bridges.

Wind induces random loads on a bridge deck distributed along the span which can be classified as lift force (upward or downward), drag force (along-wind) and moment or torque. These loads have a static component and a dynamic component. The static component of wind loads, which is proportional to the square of the mean windspeed and aerodynamic properties of the bridge cross section, influences the mean response of the bridge. The dynamic component of wind loads can be split into two parts – one which depends on *wind- or body-induced turbulence* (aerodynamic or buffeting forces), – and one which is influenced by the *motion* of the bridge (aeroelastic forces). Both dynamic wind-load components depend on mean windspeed, wind direction and aerodynamic shape of bridge cross section. The response of the bridge is influenced by its dynamic characteristics such as mechanical stiffness, mechanical damping and support conditions. Depending upon the dynamic characteristics of a bridge, wind loads can affect individual elements or the structure as a whole. Vortex shedding around individual elements such as hangers or cables can induce vibration and consequent fatigue failure of one or more of these components. Buffeting wind loads can cause horizontal, vertical or torsional vibrations of bridges. While both vortex-induced and buffeting-induced vibrations could be annoying to the traffic and pedestrians and cause fatigue in the structural members, flutter (aeroelastic instability) could cause total failure of the bridge. Wind loadings could be also critical during construction; therefore, should be considered. The aerodynamic properties of a cross section can be significantly altered by adding appurtenances such as a traffic divide or a fence or a sound barrier or a traffic sign and needs careful consideration. By identifying the expected wind loadings and the dynamic characteristics of the bridge, it can be designed or altered to mitigate undesirable dynamic responses.

Analytical assessments along with wind-tunnel experiments are required to determine possible static and dynamic responses of a bridge under wind loadings.

The types of wind effects that are usually considered in the design stage are as follows:

- static wind loading (the steady state response to the mean wind)
- buffeting (dynamic response to turbulence in the wind)
- vortex shedding by the deck

* aeroelastic instability (flutter) of the deck
* vibration of hangers or cables due to vortex shedding

For a thorough check of possible problems, some of these analyses are carried out for a bridge in a critical stage of its construction as well.

The design process includes both experimental and analytical studies. Wind-tunnel experiments are performed on models of the bridge in a wind tunnel to verify whether or not there are potential problems regarding vortex shedding, buffeting, and/or aeroelastic instability (flutter). Analytical studies include finite element computations of the static response of the bridge to wind loadings, determination of the lowest natural frequencies and modes of vibration, and computation of the bridge's dynamic response to wind buffeting and vortex shedding based on wind-tunnel results.

3.2 Current Standard of Practice

TxDOT uses the code of the American Association of State Highway and Transportation Officials (AASHTO 1992) in design practice. Following are excerpts from AASHTO (1992), section 3.15, criteria used for wind loads.

The wind loads consist of uniformly distributed loads applied to the exposed area of the structure. The exposed area is a sum of the areas of all members, including floor system and railing, as seen in elevation at 90 degrees to the longitudinal axis of the structure. Wind loads are specified for a base windspeed of 100 mph. For superstructure design, a wind load of 75 psf for trusses and arches and 50 psf for girders and beams is specified for Group II and Group V loadings. The loads shall be applied horizontally at right angles to the longitudinal axis of the structure. The total force shall not be less than 300 pounds per linear foot in the plane of the windward chord and 150 pounds per linear foot in the plane of the leeward chord on truss spans, and not less than 300 pounds per linear foot on girder spans. Loads can be reduced or increased for Groups II and V loading cases in the ratio of the square of the design wind velocity to the square of the base wind velocity provided that the maximum probable wind velocity can be ascertained with reasonable accuracy, or provided that there are permanent features of the terrain which make such changes safe and advisable. Group III and Group VI loadings comprise of loads used for Group II and Group V loadings reduced by 70 percent and a load of 100 pounds per linear foot applied at right angles to the longitudinal axis of the structure and 6 feet above the deck as a wind load on live load. For definition of different groups of loading and more details the reader is advised to refer to AASHTO (1992).

The loads specified by AASHTO (1992) specifications are static loads only which are used to include dynamic effects as well. In Sarkar et. al. (1994b) it is reported that the buffeting analysis of the tied-arch bridges on US 59 show that the static plus peak dynamic response is 1.1 to 1.8 times the static response obtained with AASHTO design loads. Moreover, AASHTO (1992) can not account for special aerodynamic phenomena such as vortex shedding and flutter.

3.3 Analytical Procedures

Analysis of bridge response due to wind involves mainly two constituents, namely, the wind characteristics and the bridge aerodynamic characteristics.

The natural wind is of boundary-layer type. The wind characteristics of a boundary-layer wind such as mean wind-speed profile, wind turbulence profile, directionality of wind are influenced by factors listed as follows:

- Geographic location of the bridge. Is it located near a coastline or a waterfront or inland?
- Surrounding terrain features. Does it have special terrain features like hills or escarpment nearby ? Is it above a water channel or a freeway ? What are the types of surrounding man-made structures ?

The bridge characteristics such as natural frequencies of vibration, mechanical damping, aerodynamic parameters lift and drag, aerodynamic damping and stiffness influence the following factors:

- Span and width of the bridge
- Shape of the bridge deck – box girder, slab, open-lattice truss
- Support conditions – cable stays, suspension, arch, girder
- Fences or sound barriers – porosity, height, shape
- Material – concrete, steel, composite
- Appurtenances – road divide, traffic sign, etc.

3.3.1 Windspeed and Wind Spectra

The terrain over which the wind approaches the bridge determines the windspeed and turbulence levels. The terrain is classified as Exposures A, B and C in ASCE 1990. Each terrain has a typical wind profile associated with it. The design windspeed is chosen from a wind-speed map given in the design code (ASCE 1990). It is taken as being able to approach the bridge from any possible direction, and the worst case of its approaching the bridge is taken normal to the broadside in most of the calculations. Special terrain features such as hills, escarpments, water or traffic channels should be considered because these alter the local wind profile.

Empirical formulae of the wind spectra $S_{uu}(z,n)$ for the along-wind turbulence component u and $S_{ww}(z,n)$ for the vertical-wind turbulence component w at any height z are assumed as follows (Simiu and Scanlan 1986):

$$\frac{nS_{uu}(z,n)}{u_*^2} = \frac{200f}{(1+50f)^{5/3}}, \quad \text{(Kaimal Spectrum)} \tag{3.1}$$

$$\frac{nS_{ww}(z,n)}{u_*^2} = \frac{3.36f}{1+10f^{5/3}}, \quad \text{(Lumley and Panofsky Spectrum)} \tag{3.2}$$

where $f = nz/U$, n = frequency in Hz, z = height in ft, U = mean windspeed in ft/s, and u_* = friction velocity in ft/s. u_* can be calculated using the mean square value of u $(\overline{u^2})$ which is equal to βu_*^2, where β is assumed (Simiu and Scanlan 1986) according to the terrain. Usually, $I_u(z) = \frac{\overline{u^2(z)}^{1/2}}{U(z)}$ = turbulence intensity of the u component is assumed for calculating u_*.

Conceptually, any other wind spectra empirical or measured, can be used instead of the above. The turbulence intensity $I_w(z)$ of the w component is taken as $0.3I_u(z)$. In the wind velocity calculations, the height z of the deck from the ground is used.

The design codes for wind loads are based on steady-state winds. The effects of fluctuations in the wind and dynamic characteristic of the structure are accounted for in the wind design codes through gust response factors. Since turbulence and dynamics are accounted for separately in the buffeting analysis, the gust response factor is taken as unity in the code formulae for calculating the equivalent mean windspeed for dynamic analysis. The following calculations show the derivation of the mean windspeed U in the buffeting analysis, based upon the ASCE (1990) code.

$$q_z = 0.00256K_z(IV)^2 = \frac{1}{2}\rho U^2 \tag{3.3}$$

$$P = q_z G_h C_f = \frac{1}{2}\rho U^2 G_h C_f \tag{3.4}$$

where q_z is the velocity pressure in psf at height z; K_z is the exposure coefficient, which is a function of height z and type of exposure or terrain; I is the Importance Factor, which depends upon the importance and use of the structure; V is the basic (fastest mile) windspeed in mph for a 50-year return period at 10 m (33 feet) as measured at weather stations; ρ is the air density; U is the equivalent mean windspeed; P is the design wind pressure; G_h is the gust response factor, which is a function of wind turbulence, dynamic characteristics of the structure, and total structure height, h; and C_f is an appropriate pressure or force coefficient.

An expression for U can be written using Equation 3.3, as follows:

$$U = \sqrt{\frac{2q_z}{\rho}} \tag{3.5}$$

3.3.2 Mean and Loading

Static wind loads are calculated with the following formulas:

$$L(lb/ft) = (\tfrac{1}{2}\rho U^2)BC_L \qquad \text{for the lift force, L} \qquad (3.6)$$

$$D(lb/ft) = (\tfrac{1}{2}\rho U^2)BC_D \qquad \text{for the drag force, D} \qquad (3.7)$$

$$M(ft-lb/ft) = (\tfrac{1}{2}\rho U^2)B^2C_M \qquad \text{for the moment, M} \qquad (3.8)$$

where ρ is the mass density of air (1.23 Kg/m^3 or 0.002378 slugs/ft^3), B is the width of the bridge deck, and C_L , C_D and C_M are the lift, drag and moment coefficients for the deck. The values of C_L, C_D and C_M are found using the wind tunnel or are extracted from the literature (e.g., Blevins 1984) for initial mean load estimates.

3.3.3 Natural Frequencies and Modes of Vibration

The natural frequencies and mode shapes of bridges are extremely important in influencing their dynamic responses to wind loading. Also, this information is used in the calculations of dynamic response. In the response behavior, different modes come into play in different ways. Several of the lower modes are expected to participate in the response of a bridge. For vortex shedding and aeroelastic instability of the bridge, the most important modes are the lowest vertical and torsional ones.

Since the lowest frequencies of each bridge lie in the region of the wind spectrum where, from a frequency content standpoint, the spectral values are decreasing rapidly (Simiu and Scanlan 1986), higher modes can be expected to have lower contributions to the deflections, as is typical of most structures. Accordingly, the first ten frequencies of the completed bridge are determined. The symmetric or anti-symmetric modes about midspan are identified. In fact, modes with frequencies above 5 Hz are not likely to be excited by the wind. A key point is that both vertical and along-wind horizontal spectra have negligible values above 5 Hz compared to the peak values below one Hertz. Thus, determining the first ten frequencies is appropriate for the analysis. Similar data for the frequencies and modes of the partially completed bridge can be computed.

An attempt will be made to classify a bridge as semi-flexible or flexible with respect to wind loading. The following definition is a fuzzy one. The structure may be considered semi-flexible if the lowest three frequencies of vibration lies between 1 Hz and 2 Hz and flexible if the lowest two or three frequencies are less than 1 Hz.

3.3.4 Dynamic Wind Loading

Generally, semi-flexible and flexible bridges are subjected to three types of dynamic wind effects: aeroelastic instability, vortex shedding, and buffeting.
Buffeting is defined as the unsteady loading of a structure due to velocity fluctuations in the oncoming flow.

Flutter or aeroelastic instability describes an exponentially growing response of the bridge deck, where one or more modes participate at a particularly critical wind velocity, possibly resulting in failure due to over-stressing of the main structural system. This phenomenon could potentially occur only in flexible bridges.

Vortices are shed from the deck at certain frequencies f_s at different mean windspeeds U according to the Strouhal number $St = f_s D / U$, where *D* is a characteristic dimension perpendicular to the flow. When the frequency of vortex shedding matches one of the natural frequencies of the deck, the vortices excite that particular mode of vibration. Vibration at this windspeed is called "lock-in." The first two vertical modes of vibration are most susceptible to vortex shedding because they have the lowest frequencies.

BUFFETING LOADS

Buffeting forces act on a bridge deck because of fluctuations in the windspeed, i.e., wind turbulence. These forces are also influenced by turbulence induced by the bluff[3] body itself. To account for the body-induced turbulence, an aerodynamic admittance function $\chi^2(K)$, is needed for each of the three forces, i.e., the lift, moment, and drag forces. In general, the admittance functions vary with the reduced frequency $K = \omega B / U$, where ω is the frequency in radians per second, *B* is the width of the deck, and *U* is the local windspeed. The admittance functions are determined in the wind tunnel. In the absence of data, one can assume a certain form of the admittance functions, for e.g. "Sears" Function, based upon previous work in the literature.

Auto-spectra of the lift, moment, and drag forces at any location, *x*, along the span of the bridge, neglecting cross-spectral components of velocity, can be obtained from the previously defined quantities and the auto-spectra of the longitudinal and vertical wind fluctuations. Denoting the longitudinal (*u*) and vertical (*w*) wind spectra by $S_{uu}(x,K)$ and $S_{ww}(x,K)$, respectively (Scanlan, 1988), the lift (L), moment (M) and drag (D) force auto-spectra are as follows:

$$
\begin{aligned}
S_{L,L}(x,K) &= (\tfrac{1}{2}\rho U^2 B)^2 \, [4C_L^2 \frac{S_{uu}(x,K)}{U^2} + (C_L' + C_D)^2 \frac{S_{ww}(x,K)}{U^2}](\chi^L(K))^2 \\
S_{M,M}(x,K) &= (\tfrac{1}{2}\rho U^2 B^2)^2 [4C_M^2 \frac{S_{uu}(x,K)}{U^2} + (C_M')^2 \frac{S_{ww}(x,K)}{U^2}](\chi^M(K))^2 \\
S_{D,D}(x,K) &= (\tfrac{1}{2}\rho U^2 B)^2 \, [4C_D^2 \frac{S_{uu}(x,K)}{U^2} + (C_D')^2 \frac{S_{ww}(x,K)}{U^2}](\chi^D(K))^2
\end{aligned}
\qquad (3.9)
$$

where $C_L' = dC_L / d\alpha$, $C_M' = dC_M / d\alpha$, and $C_D' = dC_D / d\alpha$, where α is the angle of attack; *B* is the structural dimension; and $\chi^2(K)$ is the aerodynamic admittance function.

[3]As opposed to a streamlined body where the flow remains attached to the surface of the body at large, a bluff body has blunt edges and sharp corners forcing the flow to separate out at or near the leading edges.

Another form of these force auto-spectra involving two admittance functions instead of one as in Equation 3.9 has been suggested by Sarkar (1992) and Scanlan (1993).

Wind forces on different points of a structure are only partially correlated. It is known that the spatial correlation of turbulence in the wind reduces with an increase in distance between two points (x_1 and x_2) along the span. Here the form of the spatial cross spectrum for each point along the span is assumed as in Simiu and Scanlan (1986):

$$S_{Q,Q}(x_1,x_2,K) = S_{Q,Q}(x,K)\exp(-CK|x_1-x_2|/(2\pi B)) \tag{3.10}$$

where C is the incoherency coefficient or exponential decay coefficient, which is normally assumed to lie between 8 and 16 for wind, and Q stands for L, M, or D. However, it is known that buffeting forces are better correlated than the windspeed itself (Davenport et al., 1992). Therefore, a value of C less than or equal to 8 can be assumed for the partially correlated time-domain forces presented herein. C equal to zero is taken for fully-correlated wind.

Analysis can be carried out in time domain or frequency domain. The frequency-domain approach is used commonly for wind studies but the time-domain approach has certain advantages. In the time-domain approach, peak dynamic deflections and stresses can be generated with the same finite element model of the bridge which is used for calculating the natural frequencies. However, this approach does not capture the aeroelastic effects. In the frequency-domain approach, peak responses must be estimated on a statistical basis.

TIME-DOMAIN ANALYSIS

Partially-correlated time histories of buffeting forces are generated for the time-domain analysis using the form of the cross-spectrum in Equation 3.10 with C equal to a certain assumed or measured value. In principle, there are a number of methods available to generate the time histories. In one of the methods, the time history at point j along the span, $f_j(t)$, is digitally simulated as follows (Shinozuka et al., 1972):

$$f_j(t) = \sum_{m=1}^{j}\sum_{l=1}^{N} |H_{jm}(\omega_l)| \sqrt{2\Delta\omega}\ \cos\left[\omega_l t + \theta_{jm}(\omega_l) + \phi_{ml}\right] \tag{3.11}$$

where N is the number of points in the specified target cross-spectral buffeting force matrix, $\mathbf{S^o}(\omega)$, which is real; H_{jm} are elements of $\mathbf{H}(\omega)$, a lower triangular matrix computed from $\mathbf{S^o}(\omega)$ by the matrix relationship $\mathbf{S^o}(\omega) = \mathbf{H}(\omega)\mathbf{H}(\omega)^T$; $\omega_l = l\Delta\omega$; $\Delta\omega = \omega_u/N$; ω_u is the upper cut-off frequency beyond which the power spectral density may be assumed to be zero; $\theta_{jm}(\omega_l) = \pi/4$ because $\mathbf{H}(\omega)$ is real; and ϕ_{ml} are random phase angles uniformly distributed between 0 and 2π.

FREQUENCY-DOMAIN ANALYSIS

A frequency-domain method can be used to calculate the buffeting response of the deck. Quantities used in the analysis include the equivalent design windspeed *U* calculated using Equation 3.5 of this report, the assumed spectra of along-wind and vertical-wind turbulence (Equations 3.1-3.2), the first ten modes of vibration, the aerodynamic force coefficients, the admittance functions and the flutter derivatives. The flutter derivatives influence the buffeting response by modifying the mechanical damping ratios and the natural frequencies. The force spectra are computed using the form of Equation 3.10 and using these response spectra are computed. The area under the response spectrum gives the mean-square values which are used to estimate the maximum excursion from the mean value.

The peak response can be calculated as follows. The probability that the response of the deck lies within 3.5σ bounds of its mean is 0.9998, if the probability distribution function (PDF) is assumed to be a normal distribution. In the absence of knowledge of the PDF, usually the Chebyshev inequality is used, which states that the probability of occurrence of any variable X, whose mean is m_x and standard deviation is σ_x, within $m_x - c\sigma_x$ and $m_x + c\sigma_x$ bounds is $1 - 1/c^2$. If c is taken as 3.5, then the probability becomes 0.92. Therefore, there is a 92 % probability for the response to be within the following bounds:

$$
\begin{aligned}
h_{\max} &\le \bar{h} + 3.5 \times \sigma_h^{\max} \\
\alpha_{\max} &\le \bar{\alpha} + 3.5 \times \sigma_\alpha^{\max} \\
p_{\max} &\le \bar{p} + 3.5 \times \sigma_p^{\max}
\end{aligned}
\tag{3.12}
$$

where $\bar{h}$, $\bar{\alpha}$ and $\bar{p}$ are the mean vertical, torsional and lateral deflections, respectively. The wind-tunnel experiments for buffeting involve determination of the static aerodynamic force coefficients of drag, moment and lift as well as the slopes of these coefficients with a variation with the angle of attack, α. The model is rigidly fixed to the force balance and steady state drag, moment and lift forces are measured for different windspeeds and different angles of attack to achieve this objective.

Admittance functions are determined in the wind tunnel using the static model by measuring the fluctuating aerodynamic forces under a turbulent wind. Turbulence in the wind is generated with passive devices such as grids or barriers or active devices such as oscillating airfoils or oscillating grids.

One of the most significant known results is the degree to which the fully correlated wind causes larger responses than the partially correlated wind. The selected dynamic deflections and stresses obtained with the fully correlated wind are consistently about 50 to 65 percent larger than with the partially correlated wind. These results show that the simpler model of a fully correlated wind, if used in design, could be quite conservative.

FLUTTER INSTABILITY

Flutter instability describes an exponentially growing response of the bridge deck in which one or more modes participate at a particular critical wind velocity resulting in failure due to overstressing of the main structural system. Flutter instability of the bridge can be assessed using a set of flutter-derivative coefficients calculated from wind-tunnel experiments. The flutter derivatives are dimensionless coefficients which are functions of reduced frequency $K=\omega B/U$, where U = mean windspeed, ω = frequency in rad/s, and B = deck width. The levels of aeroelastic damping and aeroelastic stiffness due to the wind-deck interaction depend on these coefficients, which are strictly functions of the shape of the cross section and hence, can be obtained only through wind-tunnel testing. Since the first few modes are uncoupled, there is a possibility of having the aeroelastic damping drive the deck to flutter instability, i.e., damping-driven flutter.

The modified damping-driven flutter criterion is as follows (Scanlan 1978, Scanlan and Jones 1990):

$$H_1^*(K)G(h_i,h_i)+H_2^*(K)G(\alpha_i,h_i)+A_1^*(K)G(h_i,\alpha_i)+A_2^*(K)G(\alpha_i,\alpha_i)\geq\frac{4\varsigma_i I_i\omega_i}{\rho B^4\omega} \tag{3.13}$$

where

$$(\frac{\omega_i}{\omega})^2=1+\frac{\rho B^4}{2I_i}\left[H_3^*(K)G(\alpha_i,h_i)+H_4^*(K)G(h_i,h_i)+A_3^*(K)G(\alpha_i,\alpha_i)\right] \tag{3.14}$$

and where $G(r_i,s_i)=\int_0^l r_i(x)s_i(x)dx$ are the modal integrals in which r_i,s_i = h_i or α_i are the vertical and torsional displacement components of the i^{th} mode shape, l is the length of the bridge, ω is the frequency in radians per second of the i^{th} mode of vibration, I_i is the generalized mass of the i^{th} mode of vibration, H_j^* and A_j^*, j=1...4 are the flutter derivatives, ζ_i is the mechanical damping ratio of the i^{th} mode of vibration, B is the deck width, and ρ is the air density.

The flutter derivative H_1^* influences the vertical damping in the vertical mode of vibration and A_2^* influences the torsional damping in the torsional mode of vibration. The flutter derivative H_4^* influences the vertical stiffness in the vertical mode of vibration and A_3^* influences the torsional stiffness in the torsional mode of vibration. H_2^*,H_3^*,A_1^*,A_4^* influence the aeroelastic coupling between the vertical and torsional modes of vibration. The flutter derivatives associated with the lateral mode of vibration are usually neglected for semi-flexible bridges.

The experiments for determining aeroelastic instability are conducted for vertical and torsional degrees of freedoms. The time history of the decaying response of the model are recorded by releasing the model with an initial amplitude at a certain windspeed. The recorded data are then used to calculate the modified damping and stiffness of the model due to the wind flow. These values of damping and stiffness are used to calculate the flutter derivatives of the model (Scanlan 1978). The most important ones are A_2^* and H_1^* which are functions of dimensionless windspeed, *K*. The experiments are carried out at several windspeeds to generate the flutter-derivative curves, thus revealing, among other things, the windspeed at which one of the derivatives may change sign, i.e. negative damping. A_1^*, A_4^*, H_2^*, and H_3^* play a role in the response of the bridge when coupling between the vertical (*h*) and torsional (*a*) deflections occur. The values of the latter flutter derivatives are not usually determined unless there is significant coupling of modes.

VORTEX-INDUCED RESPONSE OF THE DECK

Vortices are shed from the deck at certain frequencies (f_S) and at different mean windspeeds (U) according to the Strouhal number, which is defined for any cross section as follows.

$$St = f_S D / U \tag{3.15}$$

where *D* is a characteristic dimension perpendicular to the flow. The wind tunnel experiments on the section model help to determine the Strouhal number. When the frequency of vortex shedding matches one of the natural frequencies of the deck, the vortices will excite that particular mode of vibration. Vibration at this windspeed is called "lock-in."

The amplitude of vibration at the lock-in windspeed can be calculated using Equations 3.16 and 3.17:

$$Y(x) = D\xi_0 \Phi(x) \tag{3.16}$$

and

$$\xi_0 = \frac{2}{\sqrt{\varepsilon}} \sqrt{\frac{\phi_2}{\phi_4}} \left[1 - \frac{4\pi m \zeta}{\rho D^2 Y_1} St \right]^{1/2} \tag{3.17}$$

where $\Phi(x)$ is the mode shape, $\phi_2 = \int \Phi^2(x) \frac{dx}{L}$ and $\phi_4 = \int \Phi^4(x) \frac{dx}{L}$, ζ is the critical damping ratio, ρ is the air density, *m* is the mass per unit length, and Y_1 and ε are experimentally obtained parameters.

The vortex-shedding experiments are conducted at the lock-in speed, i.e., the windspeed at which the frequency of vortex shedding is equal to the natural frequency of the model. The time history of the response is recorded at the lock-in speed and the parameters of vortex shedding Y_1 and ε are identified from the data.

VORTEX-SHEDDING ANALYSIS

The lock-in windspeeds at which the first two modes may be excited are calculated using the natural frequencies obtained from the finite-element study. The lock-in windspeed for the higher modes is usually unrealizable. To use equations 3.16 and 3.17, values of the two parameters Y_1 and ε must be known for the deck. These parameters depend on the deck shape and the mechanical damping ratio ζ and are obtained experimentally from the wind-tunnel test, as indicated above. The standard deviation of a sinusoidal response of amplitude A is calculated as $A/\sqrt{2}$. This standard deviation should be added to the mean vertical lift displacement to get the total excursion as in the buffeting analysis.

VORTEX-INDUCED RESPONSE OF DECK HANGERS OR CABLE STAYS

The susceptibility of each cable used as a hanger or cable-stay to vortex-shedding excitation is examined. The procedures for calculating the natural frequencies and mode shapes of the cable, the lock-in windspeeds, and the amplitudes of vibration are given first. It is found that motion due to vortex-shedding will take place only if the critical damping ratio of the cable is below a certain level.

The formula for calculating the frequency of vibration ω (rad/s) of a cable having mass per unit length m and length L and carrying a tensile load T is given by

$$\omega = \frac{n\pi}{L}\sqrt{\frac{T}{m}}, \qquad n= 1,2,3.... \text{ mode number} \qquad (3.18)$$

The above formula is modified if the flexural rigidity EI of the cable is included in the calculation of frequency. The modified formula is

$$\omega = \frac{n\pi}{L}\sqrt{\frac{T}{m} + \frac{n^2\pi^2 EI}{mL^2}}, \qquad n= 1,2,3.... \text{ mode number} \qquad (3.19)$$

When EI is negligible, Equation 3.19 takes the same form as Equation 3.18. The corresponding mode shapes of the cable are

$$\Phi_n(x) = A \times \sin(\frac{n\pi x}{L}), \qquad n= 1,2,3.... \text{ mode number} \qquad (3.20)$$

The Strouhal number (St) of a circular cross section is 0.2 for Reynolds numbers (Re) from 500 to 10^4. Usually there are multiple cable stays or hangers in a side-by-side configuration. The ratio of the distance between cables (E) to the diameter of each cable (D) is calculated. It is known that as long as E/D is greater than 4.0, the vortices shed from one cable will not interfere from those shed by the other cable (Blevins 1984). Hence, in the calculation only one cable can be considered if E/D is greater than 4.0.

ANALYSIS

The amplitudes of vibration of the different cables at the lock-in windspeed can be calculated using Equations 3.16 and 3.17. If it is assumed that vortices are fully correlated over the entire length of the hanger in Equation 3.16, then $\phi_2 = 0.500$ and $\phi_4 = 0.375$. The parameters Y_1 and ε are taken as 4.96 and 624.0, respectively, for a circular cross section (Goswami 1991). Assuming a certain damping ratio ζ , Equation 3.16 gives the value of the amplitude of steady-state vibration Y_{max} at mid-height of the hanger. The same calculation can be repeated by assuming that vortices are correlated only over the middle third of the length of the hanger.

3.4 Wind-Tunnel Techniques

3.4.1 Full-Model Tests

A scaled version of the full model of the bridge is built and tested in the wind tunnel to study the three dimensional effect of the wind. The surrounding terrain is also modeled. This technique is very expensive and time consuming. It requires a relatively large wind-tunnel section. Therefore, it is not recommended if there are financial and time constraints. Section models can model two dimensional wind effects only but are quite accurate in predicting unusual aerodynamic behavior. Bridge buffeting, which is the effect of wind turbulence, may be sometimes overpredicted with section models.

3.4.2 Taut Strip Models

This technique was first introduced by Davenport (1972) and since then has been in extensive use by Davenport and his co-workers (Davenport et. al. 1992). This is an alternative to full model tests to examine three dimensional response characteristics of simulated turbulent wind. Tanaka (1992) points out that this method has certain advantages of the full model tests such as inclusion of three dimensional characteristics, availability of testing against an oblique wind, etc., and yet is still much simpler in concept than the full bridge model and shares the advantages of section models of low cost and short lead time. The disadvantages of this method are some technical difficulties associated with the center of rotation, frequency ratio, lateral sway characteristics, adjustment of generalized mass and structural damping, maintaining accuracy in model configuration, and requirement of relatively large wind-tunnel sections.

3.4.3 Section Models

Section models have been in use for several years both in classical and bridge aerodynamics. Scanlan and his co-workers (Sarkar et. al. 1994) have used this technique more than others in bridge aerodynamics. It has certain advantages such as low cost, short testing time, ease of model building, and requirement of relatively small wind-tunnel sections, etc. The main disadvantage of section model testing is that it can not model three dimensional wind effects including oblique winds and special terrain conditions like the full model tests. However, these effects can be accounted to a certain extent in the analytical procedure following section-model testing. Section models are debated to be more conservative than other testing methods. Section models are geometrically scaled version of the deck representing one section of the bridge span. A section model of the prototype bridge deck is built with a certain geometric scale. This choice of geometric scale is based upon the dimensions of the prototype and practical considerations associated with building section models. The material is carefully chosen to keep it light but stiff. The length of the section model is chosen as to utilize most of the wind-tunnel width. Architecture details are added as much as possible. The end plates are made of aluminum and are attached at the ends of the section model to ensure two-dimensionality of the wind flow. Necessary accessories are attached at the ends to give the model two degrees of freedom, namely vertical and torsional (rotational) motions

.

The experiments are conducted in a wind tunnel test section which is dedicated to section-model testing. The test section is usually equipped with all the necessary instrumentation to record aeroelastic and aerodynamic forces. The section should have a glass wall on one side to provide visibility and a door for easy access to the model. A hot-wire mount is fixed to the inner wall of the section which can be moved along the vertical and horizontal direction.

The model suspension system and the load cell frame are fixed on the outer side of the wall. The benefit of such an arrangement is that only the model experiences the wind flow, which helps in obtaining higher accuracy of results. The suspension system is designed such that the stiffness of the model support can be changed through addition of springs or rigid bars. Provisions are made in the test section to allow movement of the model along two degrees of freedom, i.e., vertical and torsional degree of freedom, and to allow measurement of forces along the third degree of freedom, i.e., lift (vertical), moment (torsional) and drag (along-wind). It is possible to adjust the setup to a single degree of freedom or to two degrees of freedom (vertical and torsional). Another feature of the test section is an excitation mechanism which allows the operator to give an initial amplitude to the model for any experiment including dynamic response.

4. FIELD TESTING

4.1 Introduction

Field testing is required to validate any analytical or wind-tunnel results. The dynamic properties of the bridge such as natural frequencies and natural modes as estimated with finite element modeling needs to be verified. Damping is one of the most important parameters influencing the dynamic response of the bridge but is assumed in any analysis. A true estimate can be obtained only with full-scale testing. The response of the bridge to natural wind can be also measured and the values compared with those obtained analytically based upon the wind-tunnel tests. It is particularly important to monitor the response during construction stage because the dynamic properties are quite different from the completed bridge. Every analytical method has certain assumptions and hence, verification of the predictions made by any analytical method is important.

4.2 Testing During Construction and Service

The field testing of any bridge should be performed both during construction and after completion. Sometimes the construction stage could be more vulnerable than the full-bridge stage. Field testing requires instrumentation along with a data acquisition system to measure wind parameters, such as speed and direction, and bridge response. A three cup anemometer and a direction vane can be used to measure the windspeed and wind direction at the bridge site. Bridge response can be monitored by accelerometers or geophones or strain gages or a combination of these instruments. Certain instruments may be used for a short-term period to assess the dynamic properties such as modal frequencies and mode shapes while some instruments may be installed to monitor the long-term response of the bridge during and after construction. The data acquisition system typically consists of cabling, amplifiers, signal conditioning, multiplexer boards, analog to digital boards, and either a data logger or computer to control the system, collect the data and archive the data. The instruments can be monitored both remotely and manually. The data collected at the field site is usually retrieved on a magnetic media such as hard disk of a computer or an optical media. The wind velocity data along with the corresponding bridge response data allow to check the accuracy of the analytical results which are based upon wind-tunnel results. Advanced techniques such as system identification methods can be used to extract the dynamic properties of the bridge from the obtained data both during and after construction.

4.3 Experience of WERC In Full-Scale Testing

The Wind Engineering Research Center (WERC) at Texas Tech University is on the forefront in the full-scale testing of structures. Since its inception in 1989, WERC has been a pioneer in the installation and monitoring of instrumentation required for full-scale testing, data acquisition, data validation, and data archiving, using the state-of-the-art technology. The WERC data is used world-wide for research on wind loads on low-rise buildings. WERC has an unique facility WERFL (Wind Engineering Research Field Laboratory) comprising of a meteorological tower and a low-rise building to conduct research on windspeeds and wind loads described as follows.

WERFL

The Wind Engineering Research Field Laboratory (WERFL) consists of a 160-ft high meteorological tower, a rotatable 30 x 45 x 13 ft test building, and a 10 x 10 x 8 ft data acquisition room. Figure 3 is a photograph of the tower and test building. The meteorological tower is equipped with wind-speed anemometers, wind direction and temperature sensors, and barometric pressure and relative humidity measurement instruments. These instruments provide a complete wind characterization at the field site. The test building can be rotated in a full circle on a track which is embedded in a concrete pad. The wind-induced pressures at different locations on the roofs and walls of the building are recorded by pressure taps. The pressure, windspeed and wind direction data are recorded automatically with a PC computer equipped with an A/D converter, a 20 MB removable cartridge and a data acquisition software capable of constant monitoring. The data is acquired at the rate of 30 Hz for building surface pressures and at 10 Hz for windspeed and wind direction.

LONG-SPAN BRIDGE TESTING

The Houston Baytown Bridge, which is located on State Highway 146 connecting Houston to Baytown in Texas, was recently completed and opened to the traffic in the month of September 1995. This bridge is a twin-deck cable-stayed bridge with a main span of 1250 ft and width of 78.2 ft. The first author of this report (Dr. Partha Sarkar), who is a WERC personnel, participated in the full-scale testing of this bridge one week before it was opened to the traffic. The testing was done by a team which included academicians, students and a technician led by Prof. Nicholas P. Jones, Department of Civil Engineering, The Johns Hopkins University, Baltimore (Prof. Jones was a co-advisor of Dr. Sarkar for his doctoral dissertation). The testing was done over a period of two days. It was a short-term testing because its objectives were to identify only the natural frequencies, modal damping and mode shapes of the bridge and cables. Dr. Sarkar actively participated in installing the accelerometers along the bridge span, laying of the cables connecting the accelerometers to the data acquisition system and setting up of the instrumentation system. The initial data acquisition and analysis was done on-line with the help of a spectrum analyzer. The natural frequencies of the bridge could be successfully identified on-line. The identification of modal damping and mode shapes required further data analysis.

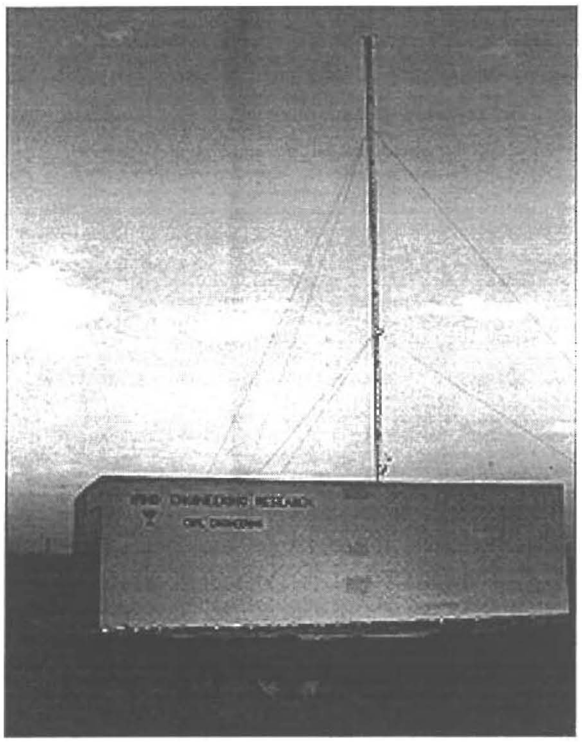


Figure 3: PHOTOGRAPH OF THE WERFL, TTU SITE SHOWING TEST BUILDING AND METEOROLOGICAL TOWER

5. CONCLUSION

The sensitivity of four overpass tied-arch bridges to wind and traffic loadings were assessed in this project. Extreme wind loading can result from hurricane storms approaching Houston from the Gulf of Mexico or from thunderstorm winds; the resulting winds are usually designated as *straight winds*. The third type of extreme wind loading can occur due to tornadic winds. In this project, the bridges have been assessed for straight winds. The probability of an intense tornado occurring the bridge site is extremely low, and hence for economic reasons the bridges are not designed to resist such an event. Analytical and wind-tunnel studies were conducted to check the vulnerability of the bridge decks to aerodynamic phenomena such as flutter, buffeting and vortex shedding. The results of this study were reported in a report (Research Study No. 7-1982-2, December 1994) submitted to TxDOT. The overall conclusion of the study was that the proposed tied-arch bridges over U.S. 59 in Houston can resist anticipated wind and traffic loadings when completed. The current report briefly discusses the wind climate for the Houston area and analytical methods used for assessing various wind-effect phenomena associated with the bridges. The discussion in this report is general and is meant to provide guidance in future bridge designs for wind effects. it can be used for gaining preliminary knowledge of the dynamic effects of wind on flexible or semi-flexible bridges.

6. RECOMMENDATIONS

Any laboratory and analytical procedure requires validation in the field. Wind-structure interaction problems are very complex. Hence, even the best analytical solution based upon accurate wind-tunnel testing could differ from the actual behavior under service conditions. The wind analysis in this project is based upon a combination of laboratory wind tunnel and finite-element modeling of the bridge structure. The analytical procedure of finite-element modeling depends upon many assumed parameters such as strength of concrete, boundary conditions, etc. Deviation of these parameters from the specified design values can cause results of the finite element to differ from the actual values. In addition, wind tunnel studies are based on simulated winds. The characteristics of winds at the bridge site can be quite different because of surrounding terrain. It is, therefore, recommended to pursue field measurements on the bridges as originally planned in the project. Limited measurements can reveal the quality and variation in construction and help validate the analytical and wind-tunnel results. It is recommended that the field measurements are done both during construction and after the bridges are completed. Valuable data of the windspeeds collected at the bridge site along with bridge response during a moderate (say 40 mph) to high (say 80 mph) wind event can assist in future design of tied-arch, suspension or cable-stayed bridges of longer spans.

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