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A Review of the Susceptibility of the Scalpay Bridge to Aerodynamic Effects.

by

R.A.McD. Galbraith, J. Anderson and M. Vezza.

of

Department of Aerospace Engineering
University of Glasgow
Glasgow G12 8QQ.

on behalf of

Crouch, Hogg and Waterman, Consulting Civil,
Structural and Environmental Engineers.

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Summary

An independent review is presented of the procedures employed by Crouch, Hogg & Waterman (CHW) in their assessment of the likely aerodynamic effects on the proposed Scalpay Bridge. The review identifies the principal design criteria relevant to the aerodynamic and structural dynamic performance of the Scalpay Bridge as prescribed in BD 49/93. On the basis of data and information supplied by CHW, an assessment is made of the degree to which these criteria are satisfied. It is concluded that there are several sensitive areas in the design analyses undertaken by CHW that should be reconsidered in view of the apparent susceptibility of the bridge to aerodynamic effects. In particular, it is recommended that the response of the bridge to vortex excitation and turbulence, and the narrow stability margin against galloping, be investigated further.

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1. Introduction

Following preliminary discussions in May, 1995, the consultants for the Scalpay Bridge, Crouch Hogg and Waterman (CHW), commissioned the authors to review their assessment of the likely aerodynamic effects on the proposed bridge to cross from North Harris to the island of Scalpay. A summary of the assessment by CHW, along with the consultants' instructions, is contained in Appendix A.

The proposed Scalpay Bridge has a long, slender central span of box girder construction. The cross-section appears as an effective rectangular cylinder and so, in all winds, will be subjected to fluctuating forces which may excite the bridge into severe wind induced motions. Procedures for the assessment of unsteady aerodynamic effects on bridges are contained in BD 49/93 *Design Rules for Aerodynamic Effects on Bridges.*

The present report is a review of the application of the design standard BD 49/93 by CHW in the context of the Scalpay Bridge. The review illustrates the various steps and decision points of BD 49/93 together with the effective path taken by CHW (see Fig. 1), and provides a commentary on the various aspects of the decision process and associated design calculations. This commentary takes the form of explanatory notes highlighting specific aspects of each phase of the design assessment such as the local wind environment, structural dynamic characteristics, and aeroelastic stability and response.

2. Assessment of Aerodynamic Effects Based on BD 49/93

In this section, reference is made to relevant data and supporting information supplied by CHW. In general, this data comprised design calculations associated with BD 49/93 (natural frequencies, vortex excitation and limited amplitude response, turbulence response, galloping and flutter instability criteria, etc.) and calculations based on the additional design codes :

BD 37/88	*Loads for Highway Bridges*
BS 8100	*Lattice Towers and Masts* *Part 1: Code of practice for loading* *Part 2: Guide to the background and use of Part 1*

2.1 Preliminaries

Note 1: Maximum Span
The span is less than 200 metres.

Note 2: Bridge Type
The bridge is not a footbridge.

Note 3: Minimum Span
The span is greater than 50 metres.

Note 4: Parapets
All parapets, edge members, median barriers are taken to be within the limits of the design rules (BD 49/93).

2.2 Extreme Wind Speeds

Note 5a: Design Wind Speed
A number of formulae are available for the calculation of design wind speed, and CHW have employed three of these in their various analyses (BS 5400 Part 2, BS 8100 Part 1, BD 49/93). In general the formulae are founded on the meteorological "basic" hourly wind speed, V_B, appropriately modified by a number of factors to account for some or all of the following effects:

(1) *local topography* (K_T), i.e. sheltered valleys, exposed hills etc;
(2) *terrain roughness* (K_R), affecting velocity profile and turbulence levels;
(3) *averaging time* (K_G), to obtain appropriate wind gust speed for static or aeroelastic response;
(4) *height above ground* (K_2), normally by power law representation;
(5) *statistical factor* (K_1), associated with probability of wind speed exceeding basic speed in given return period;
(6) *altitude* (K_A), i.e. height above sea level of structure;
(7) *wind direction* (K_D), which affects likelihood of exceedance;
(8) *seasonal factor* (K_S), for temporary structures only.

Factors (6)-(8) have been generally ignored in the past, but are included in the latest models (BRE [1989]). Only one of these three, (7), would have relevance here and for westerly winds $K_D \approx 1$.

Throughout the calculations the basic wind speed value employed by CHW for the location on Scalpay Sound was 36m/s. It should be noted that more recent and comprehensive analyses of wind records (BRE [1989]) has resulted in the basic wind speed being reduced to 28m/s.

The formats of the three wind speed formulae are as follows:

BS 5400: $V_{des} = K_T S_2 K_1 V_B$ where S_2 incorporates both K_2 and K_G for an exposed rural situation ($K_R = 1$).

BS 8100: $V_{des} = \gamma_v K_R K_2 K_D V_B$ where γ_v is a safety factor

BD 49/93: $V_{des} = 1.25 K_1 K_2 V_B$

BS 5400 and BS 8100 have been used for the static and mildly dynamic analyses, whereas BD 49/93 is exclusively employed for the aeroelastic assessment. A number of points have been noted concerning the implementation of the above formulae and these are detailed below.

Statistical Factor

Of particular note in all three cases is the use of $K_1 = 1$ for the statistical factor. For a given return period N, the probability P of V_B being exceeded is given by $P = 1 - (1 - p)^N$, where p is annual exceedance probability 0.02. Therefore for a return period of 120 years the probability of V_{des} exceeding V_B is 91.1% compared with 63.6% over the standard 50 year period. This seems a very high likelihood and has implications for the predicted aeroelastic responses, in particular vortex excitation and transverse galloping. This effect could be assessed by increasing V_{des} to reduce the probability, e.g. increasing the wind speed by 6.5% would reduce the probability to that for the 50 year period, i.e. using $K_1 = 1.065$.

Category of Terrain

Some ambiguity exists as to the appropriate terrain category for the bridge location in Scalpay Sound, in particular the choice between flat coastal area with off-sea wind and flat or undulating countryside. CHW's solution has been to examine the implications of employing both of these descriptions, both for mean wind and turbulence effects. This seems an acceptable way of dealing with this difficulty.

Local Topography

The situation of the bridge in a relatively narrow channel would suggest that wind funnelling could be an important factor, however no account has been taken of this in the calculations. Typically a 10% margin is incorporated in V_{des} to account for this, i.e. using $K_T = 1.1$.

Wind Speed for Aeroelastic Assessment

The BD 49/93 formula explicitly incorporates both the statistical factor (K_1) and the height factor (K_2), although the previous remarks concerning the choice of $K_1 = 1$ by CHW also apply here. The numerical factor 1.25 therefore must account for all other relevant affects, i.e. appropriate gust, topography and terrain factors.
Assuming undulating countryside, $K_R = 1$ and the terrain factor has no effect.
As mentioned above it may be appropriate to include a topography factor $K_T = 1.1$ to account for wind funnelling.

This leaves $1.25/1.1 = 1.14$ to account for wind gusting. However according to BS 5400 (and CHW) the static gust factor is approximately 1.3 (by calculating S_2/K_2).
Based on published data (Cook [1985]) the gust factor K_G and averaging time t are approximately related by $K_G = 1 + 0.42\sigma_u \ln(\frac{3600}{t})$, where σ_u is the along wind turbulence intensity (approximately 0.16 at bridge height in open country). Hence the following averaging times result:

$$K_G = 1.14 \quad t = 448\text{s} \quad (\approx 7\tfrac{1}{2}\text{ min.})$$
$$K_G = 1.3 \quad t = 41\text{s}$$
$$K_G = 1.25 \quad t = 87\text{s} \quad (\approx 1\tfrac{1}{2}\text{ min.})$$

However, BS CP3 recommends a gust duration of 15s for structures with horizontal dimension greater than 50m. This would result in a $K_G \approx 1.37$ and hence a wind speed given by $V_{des} = 1.51 K_1 K_2 V_B$. Employing the values $K_1 = 1.065$ and the expression $K_2 = 0.678 z^{0.170}$, obtained from logarithmic regression of the BS 5400 data, the design wind speed for aeroelastic assessment becomes:

For V_B = 36m/s : $\quad V_{des} = 1.51 \times 1.065 \times 1.19 \times 36 = 68.9\text{m/s}$ (154mph)

For V_B = 28m/s : $\quad V_{des} = 68.9 \times \dfrac{28}{36} = 53.6\text{m/s}$ (120mph)

Since the latter figure is based on the most recent analyses of wind speed records it is the more appropriate value. This speed corresponds closely with that employed by CHW (52.7m/s).

2.3 Bending and Torsional Frequencies

Note 5b: Estimation of Fundamental Natural Frequencies

The natural frequencies in bending have been estimated via a standard finite element analysis. The torsional frequency, however, has been estimated from the simple analytical expression provided in Annex B of BD 49/93 for box girder bridges (§3.2). The torsional frequency is expressed as a function of the fundamental bending frequency. The expression is based on simple bending/torsion theory of closed thin-walled tubes; *viz.*

$$f_T = f_B\sqrt{P_1(P_2 + P_3)}$$

where f_T is the fundamental torsional frequency, f_B is the fundamental bending frequency and P_1, P_2, P_3 are parameters dependent on the geometric and mass properties of the bridge section at the mid-span.

A simplified expression for the fundamental transverse bending frequency f_B is also provided in BD 49/93 (Annex B §2.). The validity of this expression is determined by the condition $\sqrt{I_b/w}\big|_{\text{support}} < 2\sqrt{I_b/w}\big|_{\text{mid-span}}$ where I_b is the section transverse second moment of area and w is the section weight per unit length. Provided this condition is satisfied and there is reasonable correlation between the bending frequency derived from the simplified expression and the corresponding frequency derived from the finite element analysis, the simple analytical expression for f_T above can be expected to yield an appropriate estimate of the natural frequency in torsion.

From the available data, an estimate of f_B is given by

$$f_B = \frac{K^2}{2\pi L^2}\sqrt{\frac{EI_b g}{w}}\bigg|_{\text{mid-span}}$$

with

$$K \approx 4.12,\ L \approx 171.5\text{m},\ E \approx 205\text{GN/m}^2,\ I_b \approx 1.441\text{m}^4,$$
$$w \approx 130.3\text{KN/m},\ g \approx 9.81\text{m/s}^2$$

That is,

$$f_B \cong 0.433\text{Hz}$$

This compares with a value $f_B \cong 0.678\text{Hz}$ from the finite element analysis.

The latter value is used by CHW to estimate the torsional frequency f_T. However, the parameter P_1 in the expression for f_T is related to the section polar moment of inertia of an individual box at the mid-span, I_{pj}. CHW's calculation of I_{pj} is **incorrect**. CHW's calculation relates to the polar second moment of area of the an individual box section. This error is present in the calculations of the torsional frequency both for the complete bridge and for the bridge during erection phase. A similar error appears in the calculation of the effective section polar radius of gyration used in the evaluation of the classical flutter speed (§2.1.3.3). A consequence of the error is the overestimation of the fundamental torsional frequency, f_T. It should be noted that, for the case of a homogeneous material with a constant density distribution, the error is accounted for by simply factoring the polar second moment of area by the material density. However, this correction procedure is not appropriate in the case of the effective section polar radius of gyration used in the calculation of the classical flutter speed. Here, the bridge deck and box-girder section possess different material densities and the radius of gyration must be evaluated from the polar moment of inertia for the complete section.

In the absence of appropriate geometric and mass data for the section, it is not possible to assess the full implications of this error. Nevertheless, from the available data and assuming a constant material density for the steel box-girder section of $\rho_{\text{steel}} \approx 7850\,\text{Kg/m}^3$, an estimate of the torsional frequency based on the finite element value of transverse bending frequency is given by

$$f_T \approx 9.51\,\text{Hz}$$

Alternatively, using the bending frequency based on the simplified expression for f_B, the estimate is given by

$$f_T \approx 6.08\,\text{Hz}$$

These values compare with an (erroneous) value of $f_T \approx 11.56\,\text{Hz}$ calculated by CHW.

An additional factor affecting the estimation of the torsional frequency is the bridge-deck contribution to the section polar moment of inertia as defined in Annex B (§3.2) of BD 49/93. This definition takes no account of the offset of the bridge-deck centre of mass from the centre of mass of the complete section. The effect of this offset is to increase the polar moment of inertia of the complete section and therefore reduce the value of the parameter P_1 with a resulting decrease in the torsional frequency.

The estimates of f_T presented above should therefore be viewed as upper estimates on the torsional frequency. A more accurate estimate of the fundamental natural frequency in torsion should be determined from a finite element analysis.

2.4 Vortex Excitation

When a bluff body such as the cross-section of the Scalpay Bridge is subjected to a cross-wind, then the nature of the flow is such as to create an unsteady periodic shedding of vortices from that body which trail behind in the form of a wake.

The effect of these alternating vortices and, indeed, the basic unsteady nature of the flow, is to impose upon the structure or body small periodic forces which, should the frequency of that periodicity approach any of the natural frequencies of the bridge, may excite quite severe and violent oscillations. A factor affecting the severity of these oscillations is the natural damping of the structure itself.

It is normally possible to assess, by a variety of methods, approximate values of the natural bending and torsional frequencies of a bridge structure. The problem then arises as to whether or not the frequencies associated with the shed vorticity for the assessed wind regime will be sufficiently well separated from the natural frequencies of the structure as to be unimportant. It is the purpose of this section to comment on the susceptibility of the bridge to these effects.

Note 6: Bending and Torsional Frequencies
The first bending frequency has been assessed at ≈ 0.678 Hz and so is less than the 5Hz limit defined in BD 49/93. The torsional frequency has been assessed at higher than 5Hz (see Note 5b) and so, in accordance with BD 49/93, no further consideration is required. As a consequence of the low natural bending frequency ($f_B \approx 0.678$ Hz), the bridge may be susceptible to vortex excited oscillations in bending.

Note 7: Bridge Construction
The bridge is not of truss-girder construction.

Note 8: Critical Wind Speed (Bending Mode)
In classical vortex shedding from bluff bodies, the frequency of shedding is normally given in non-dimensional form as the Strouhal number $S_t = \frac{fd}{V}$ which, for low Mach numbers, is a function only of the Reynolds number. Here, f is the frequency of

shedding of vortex pairs, d is a characteristic cross-wind dimension and V is the onset wind speed. The relationship between the Strouhal number and Reynolds number is normally obtained experimentally and, for a wide range of wind speeds, the Strouhal number is approximately constant.

If the frequency of vortex shedding coincides with the natural frequency of the structure in either bending or torsion, $f_{B/T}$, then the wind speed at which this occurs (termed the critical velocity) is given by

$$V_{cr} = \frac{f_{B/T} d}{S_t}$$

The assumption of constant Strouhal number is not always valid but for the Scalpay Bridge design where $\frac{b^*}{d_4} = 2.21 < 5$, the guidelines in BD 49/93 indicate that this is satisfactory and so

$$V_{cr} = 6.5 f_{B/T} d_4$$

From CHW's calculations, the critical velocity is given as 16.7m/s and the reference wind speed is assessed at 52.7m/s. The reference wind speed is, therefore, greater than the critical wind speed and, hence, the bridge is susceptible to vortex excited oscillations. Consequently, an assessment of the bending amplitude is required.

Note 9: Amplitude of Vibration (Bending Mode)

The (mean to peak) amplitude of vertical flexural vibrations due to vortex excitation, $y_{\max}$, is defined in §3.1.2. of BD 49/93 and is given by the expression

$$y_{\max} = \frac{b^{0.5} d_4^{\,2.5}}{4 m \delta_s} \rho$$

where δ_s is the structural damping. Sample values of structural damping are given in BD 49/93. The appropriateness of the value selected by CHW has not been verified. The maximum peak to peak vertical deflection likely to occur in the Scalpay Bridge, for the given inputs, has been calculated by CHW as 95.2mm ($\approx$ 4 inches). The effect that this peak to peak amplitude may have is assessed through a dynamic sensitivity parameter K_D which, for bending effects, is given by

$$K_D = y_{max} f_B^{\,2}$$

This sensitivity parameter equates to 22mm/s^2 and from Table 1 of BD 49/93 (Assessment of Vortex Excitation Effects) the motion discomfort that K_D will instil corresponds to 'unpleasant'.

Note 10: Critical Wind Speed (Torsional Mode)
No critical velocity for torsional oscillations has been provided and it is assumed that this is a consequence of the natural torsional frequency having been assessed at a value of greater than 5Hz.

Note 11: Calculation of Static Load Factors and Fatigue Damage
No information has been provided for the calculation of equivalent static load factors and estimates of fatigue damage. Hence no comment.

2.5 Turbulence Response

Note 12: Modal Frequency Criteria
BD 49/93 (§2.1.2) requires the dynamic effects of turbulence to be considered if either of the fundamental bending or torsion frequencies is less than 1Hz. In the present application, both the fundamental transverse and lateral bending frequencies predicted by the finite element analysis are less than 1Hz (assuming that the existing finite element model includes out-of-plane degrees-of-freedom, the finite element calculations indicate that the natural frequency in torsion is not less than 3 Hz).

In accordance with BD 49/93 (§3.3), it is therefore necessary to determine the peak amplitude turbulence response in the critical bending mode(s) under a prescribed mean hourly wind speed as defined in BS 5400 (Part 2).

Note 13 & 14: Peak Dynamic Response due to Turbulence
A variety of techniques are employed by CHW to assess the effects of gusts on the dynamic response of the bridge structure. Calculations have been performed for a range of terrain categories and reference wind speeds (see Note (5a)). The principal procedure adopted by CHW for the evaluation of the peak dynamic response to turbulent gusts follows that outlined in BS 8100 (Parts 1 and 2). Additionally, the procedures outlined in BD 37/88 are used for comparative purposes.

Equivalent Static Load Method
By introducing a dynamic amplification factor, the peak dynamic load on the structure can be expressed as an equivalent static load. This load is superimposed on

the mean load due to the mean wind speed to give the total equivalent static load. A simple procedure for the determination of the effects of gusts on bridge structures is presented in BD 37/88 (§5.3.2). This approach is likely to lead to underestimation of the dynamic effects of turbulent gusts. Nevertheless, the approach provides a useful baseline for comparison with more accurate methods based on spectral techniques.

Approximate Spectral Method

Sections C.5.6 - C.5.7 of Part 2 of BS 8100 contain a general account of approximate spectral methods. The modal generalized force response spectrum associated with turbulent fluctuating wind speeds is decomposed into a narrow-band (quasi-resonant) component and a broad-band (quasi-static) component. The narrow-band component is assumed uncorrelated with the broad-band component. The validity of this decomposition (Wyatt [1981a,b]) depends on several criteria related to the lowest natural frequency of the structure, the lateral scale of the longitudinal gust component, the relative size of the structure, and the mean (hourly) wind speed. For many practical structures these criteria are satisfied automatically. However, no formal check has been conducted by CHW for the Scalpay Bridge and its environs.

A basic assumption in the simplified form of analysis widely adopted in practice is that the modal narrow-band contributions are statistically independent. This assumption is questionable where two modes associated with perpendicular directions possess similar modal characteristics (Wyatt [1981a,b]). The fundamental transverse bending frequency predicted by the finite element analysis is $0.678\,\text{Hz}$ while the lateral bending frequency is $0.513\,\text{Hz}$. The close proximity of the natural frequencies of the transverse and lateral bending modes, and the potential spatial similarity of the modes, therefore have important implications for the analysis of the turbulence response.

For the approximate spectral approach described in BS 8100, the peak dynamic load on the structure due to turbulent wind gusts, P_{max}, is expressed as

$$P_{max} = \overline{P}(1+G)$$

where

$\overline{P}$ is the mean wind load

G is the gust factor which represents the response due to the fluctuating components of wind. This includes both the quasi-static (broad-band) response and the narrow-band (modal) response.

Procedures for the evaluation of the gust factor G are developed in BS 8100. However, BS 8100 is essentially applicable to vertical line-like structures. It is not clear that the form of the aerodynamic admittances adopted in CHW's calculations of the quasi-static and narrow-band contributions to G are appropriate for horizontal line-like structures of the type required in the present application (*cf.* expressions for J_a, J_p and J_{p_1} as per eqns(5.25)-(5.27), page 145 and eqns(5.31)-(5.33), page 146). In particular, in the case of the admittance J_p, the correlation function in the vertical direction cannot be assumed *a priori* to possess the same decay characteristics as the corresponding correlation function in the spanwise direction.

The CHW calculations appear to be concerned solely with the turbulence response in the out-of-plane (lateral or along wind) bending mode. No comparable calculations appear for the response in the in-plane (transverse or vertical cross-wind) bending mode. Furthermore, no attempt has been made to assess the effects, on the peak dynamic response, of potential correlation between the transverse and lateral bending mode responses.

Calculations are also presented for the peak response amplitude in the transverse direction. The procedure used is somewhat confusing. The calculations appear to be based on aerodynamic characteristics in the transverse (in-plane) direction but make use of the modal characteristics of the lateral (out-of-plane) bending mode!

Note 15: Combined Effects of Vortex Excitation and Turbulence

BD 49/93 (§4.) requires the combined effects of aerodynamic loading to be examined for the mode of vibration under consideration. However, the manner in which loads should be combined to meet the requirements of BD 49/93 is ambiguous.

The CHW calculations appear to have considered these aerodynamic effects for distinct modes of vibration (in-plane bending for vortex excitation and out-of-plane bending for turbulence response)! Consequently, it is not possible to comment on the adequacy of the design under the combined effects of vortex excitation and turbulence.

2.6 Galloping Oscillations

Based on a linear representation of the problem, galloping oscillations are likely to occur at all speeds above a critical threshold, and can arise as either bending or torsional oscillations.

Note 16: Bridge Type

The proposed bridge is of box girder design and therefore belongs to the category 3, 3A, 4 and 4A.

Note 17: Geometric Configuration

The numerical values for total width, b, and depth, d_4, appear to be in order and b/d_4 is indeed less than 4. Therefore according to the code the conditions for the onset of bending oscillations must be assessed.

Note 18: Transverse Galloping (Bending Oscillations)

The critical wind speed, V_g, required for the initiation of transverse galloping oscillations is obtained from:

$$\frac{V_g}{f_B d_4} = V_{R_g} = -\frac{4m\delta_s}{\rho d_4^2\left(\frac{dC_L}{d\alpha} + C_D\right)}$$

where $\frac{dC_L}{d\alpha}$ is the lift curve slope and C_D is the drag coefficient.

Comparing with the formula from BD 49/93 this implies that $C_g = -\frac{4}{\left(\frac{dC_L}{d\alpha} + C_D\right)}$.

Published information (Blevins [1977]) indicates that for a rectangular section with $\text{breadth}/\text{depth} = 2$, $\frac{dC_L}{d\alpha} + C_D = -3$ and hence $C_g = 1.33$. For the Scalpay bridge $b/d_4 = 2.21$ and $C_g = 1$ (overhang $< 0.7d_4$). Hence, the value employed seems reasonable.

The critical velocity calculated by CHW is above the predicted wind speed by 45%. The minimum margin allowable is 30% . The effect of uncertainties in any of the factors contributing to the calculation are examined below.

Effect of Bending Frequency

The discussion on bending frequency in Note 5b indicates that the appropriate value may be lower than that employed by CHW. This would have the effect of lowering V_g and hence reducing the margin of safety.

Effect of Wind Speed

The methods of assessing wind speeds are discussed in Note 5a. The alternative suggestion for calculating this speed was marginally higher than that employed by CHW, but would nevertheless further reduce the margin of safety.

Effect of Main Span Mass Distribution

BD 49/93 does not specify clearly how to calculate the mass per unit length of a bridge where this varies along the span. CHW have used the average mass per unit length, but a more representative value would be the modal mass per unit length. For the fundamental bending mode this would be weighted towards the mid-span mass per unit length, i.e. lower than a straight average. This would reduce V_g and hence reduce the margin of safety. The most severe case could be assessed by simply using the mid-span value. The amplitude of motion due to vortex excitation would also be affected (see Note 9).

Effect of Turbulence

Studies have indicated that turbulence can have a marked effect on the critical transverse galloping speed (Novak and Tanska [1974]). However for a rectangular section with shorter side facing the wind, turbulence appears to increase the critical speed and hence the margin of safety.

Given the small margin of safety predicted by CHW the possibility of additional factors lowering or even eliminating this margin should be of some concern, and further investigation may be necessary.

Note 19: Torsional Motion

The predicted critical velocity is well beyond the lower limit at which torsional divergence would be a problem, even accounting for the errors in the frequency calculation referred to in Note 5b. However the actual numerical value of the divergence speed is unrealistic as the model is no longer valid at such speeds (compressible flow aerodynamics required).

2.7 Classical Flutter

Note 20: Estimation of Bending/Torsion Flutter Speed

The criterion for bending/torsion flutter (BD 49/93 (§2.1.3.3)) is based on a low frequency approximation to the bending/torsion flutter solution of a flat plate aerofoil in two-dimensional incompressible flow. The criterion is strictly applicable to the two-degree-of-freedom case of transverse bending only with no lateral offset of the section centre of mass from the section shear centre (elastic axis) and no torsional coupling with the lateral bending motion. This latter condition is unlikely to be met in the case of a box-girder design of the type considered. Here, the section centre of mass location in the transverse direction will not, in general, coincide with the section shear centre and a degree of inertial coupling between the lateral bending mode and

the torsion mode is to be expected. However, while significant inertial coupling may have an impact on the bending/torsion flutter characteristics, in the present context the dominant mid-span sections are expected to exhibit only weak inertial coupling.

The CHW calculation for classical flutter makes use of the (erroneous) natural frequency in torsion f_T and section radius of gyration r (see Note 5b). Based on these values of f_T and r, the critical flutter speed $V_{Rf} > 2 \times 10^3$ m/s. This excessive value of V_{Rf} lies outside the range of validity of the original flutter model. Nevertheless, the revised estimates of the natural frequency in torsion indicate that the separation of the natural frequencies in transverse bending and torsion is relatively large ($f_B/f_T << 1$) and hence classical flutter is possible only under conditions of high dynamic pressure.

3. Conclusions and Recommendations

It is concluded that there are several sensitive areas in the calculations provided by CHW for the Scalpay Bridge that should be reconsidered in view of the apparent susceptibility of the bridge to aerodynamic effects.

The main areas of concern are:

- The methods of assessing the local extreme wind speeds used in the design calculation by CHW have been superceded by recent work on meteorological records. However the reference wind speed derived by CHW appears to be of the right order of magnitude for design purposes.

- The calculation by CHW of the bridge natural frequency in torsion is in error and should be re-examined. The predicted effect is to lower the natural frequency towards the limit below which torsional effects must be assessed in the case of vortex excitation. Torsional divergence and classical flutter are not predicted to present any difficulties due to this modification.

- The limited in-plane amplitude response due to vortex excitation corresponds to an experience referred to as "unpleasant" in BD 49/93. Uncertainties in structural damping and appropriate mass values could aggrevate this problem further. Wind tunnel tests would be required to identify more precisely the magnitude of in-plane vibrations due to vortex shedding.

- An inadequate assessment of the effect of turbulence on the bridge dynamic response has been carried out by CHW. In particular, the proximity of lateral and transverse fundamental frequencies and the use of vertical turbulence characteristics invalidate the approach taken.

- Although the calculations by CHW indicate that the critical galloping speed is above the limit wind speed, the margin of safety is not substantial. When consideration is given to the deleterious effects of lower bending frequency, lower mass and lower damping, the possibility of these factors eliminating this margin should be examined. For detailed information on the bridge's susceptibility to transverse galloping, wind tunnel tests would be required.

References

Blevins, R.D. [1977]
Flow -Induced Vibration, Van Nostrand Reinhold.

BRE [1989]
The Assessment of Wind Loads.
BRE (Building Research Establishment) Digest 346, BRE Publications.

Cook, N.J. [1985]
The Designer's Guide to Wind Loading of Building Structures.. Part 1. Butterworths.

Novak, M. and Tanska, H. [1974]
'Effect of Turbulence on Galloping Instability', *J. Eng. Mech. Div. ASCE* **100**, 27-47.

Wyatt, T.A. [1981a]
'The Dynamic Behaviour of Structures Subject to Gust Action', in *Wind Energy in the Eighties*, CIRIA, London.

Wyatt, T.A. [1981b]
'Evaluation of Gust Response in Practice', in *Wind Energy in the Eighties*, CIRIA, London.

Appendix A

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CROUCH·HOGG
WATERMAN
Consulting Civil, Structural and Environmental Engineers

The Octagon
36 Bothwell Street
Glasgow G2 6EE
Telephone 0141 552 2000
Fax 0141 552 2626

Department of Aeronautical Engineering
James Watt Building
University of Glasgow
GLASGOW
G12 8QQ

For the attention of Professor Galbraith

Dear Sirs,

SCALPAY FIXED LINK
SCALPAY BRIDGE
AERODYNAMIC STUDIES

We have been appointed by Western Isles Islands Council to design a major steel box girder across the Sound of Scalpay in the Western Isles. We have enclosed a location plan together with a General Arrangement drawing of the proposed bridge for your information.

The bridge is susceptible to aerodynamic effects and we have taken account of this in our design. We are now therefore looking to appoint an independent body to review both the susceptibility of the structure to aerodynamic effects and our approach in quantifying these effects and have enclosed background notes together with the proposed remit of this review for your consideration.

In the first instance we would be grateful if you would provide a cost estimate for undertaking this review. Please do not hesitate to contact our Messrs Johnston or Salisbury if you have any queries or if you wish to meet to discuss these matters further. Your earliest response would be greatly appreciated.

Yours faithfully

for and on behalf of
Crouch Hogg Waterman Limited

Encl

1. Introduction

Crouch Hogg Waterman were commissioned by Western Isles Island Council to design a bridge across the Sound of Scalpay linking the Island of Scalpay with the main island of Harris/Lewis.

The preferred scheme for bridging the sound has a mainspan of 170.0m with sidespans of 70m and 51.9m respectively and the form of the bridge comprises a composite steel box girder deck of varying depth with integral inclined steel box girder legs. (Refer to the General Arrangement Drawing No. 12972(04)50). The bridge runs almost due north-south and is very exposed to east and west winds.

The bridge has been designed for static and aerodynamic effects of wind load to Department of Transport bridge directives BD 37/88 and BD 49/93 respectively.

In assessing the bridge for aerodynamic effects certain aspects of the structure and its environment make it unique.

Firstly it is subject to a maximum 3 second gust of 56m/s which is the highest of any major bridge in Great Britain.

Secondly as the bridge is only 8.4m wide it is relatively slender in plan and the aspect ratio of breadth to depth is lower than any other comparable bridge in the British Isles.

2. Bridge Aerodynamics

Design of the bridge for aerodynamic effects has been undertaken in accordance with BD 49/93 "Design Rules for Aerodynamic Effects on Bridges" under the following headings:

i) Limited Amplitude Response
ii) Divergent Amplitude Response
iii) Non-oscillatory Divergence

The bridge was found not be susceptible to divergent amplitude and non-oscillatory effects but was found to be susceptible to limited amplitude effects as follows:

The effects of vortex excitation were quantified using both formulae given in BD 49/93 and by running an harmonic response analysis using the computer programme LUSAS Version 11.0. Comparable amplitudes of about 60mm were obtained from both methods for the first bending frequency of 0.67Hz.

The effects of turbulence response were firstly quantified by undertaking a dynamic analysis using LUSAS. Because of uncertainties in the derivation of the input for this analysis alternative approaches were considered.

Reference was made to BS 8100 Parts 1 and 2 the British Standard for Lattice Towers and Masts together with a paper from a CIRIA conference entitled "Wind Engineering in the Eighties".

The approach adopted in both of these references is to derive an equivalent static gust factor.

Good correlation was obtained between the deflections derived from a static analysis using the equivalent gust factors and the amplitudes of vibration obtained from the dynamic analyses.

Summary

We have considered the aerodynamic susceptibility of Scalpay Bridge using the published standards and technical memoranda and have designed the structure for the consequent effects of limited amplitude response.

The bridge is however unique in terms of its geometrical proportions and the maximum wind loads to which it will be subjected and we therefore would wish to have the methods and techniques we have utilised reviewed in terms of providing a reliable engineering solution.

To address this problem we have recommended to the client that an independent body be appointed to carry out a review of the work we have undertaken to date on the basis of the following remit and to comment generally on the likely aerodynamic response of the structure based on published information.

a) Using all relevant data and other supporting information supplied by ourselves to review the approach we have taken in the application of the rules and guidelines given in the relevant technical memorandum, British Standards and published papers.

b) To review the applicability of these rules and guidelines in this context.

c) To highlight any omissions or areas where further studies or investigations are required.

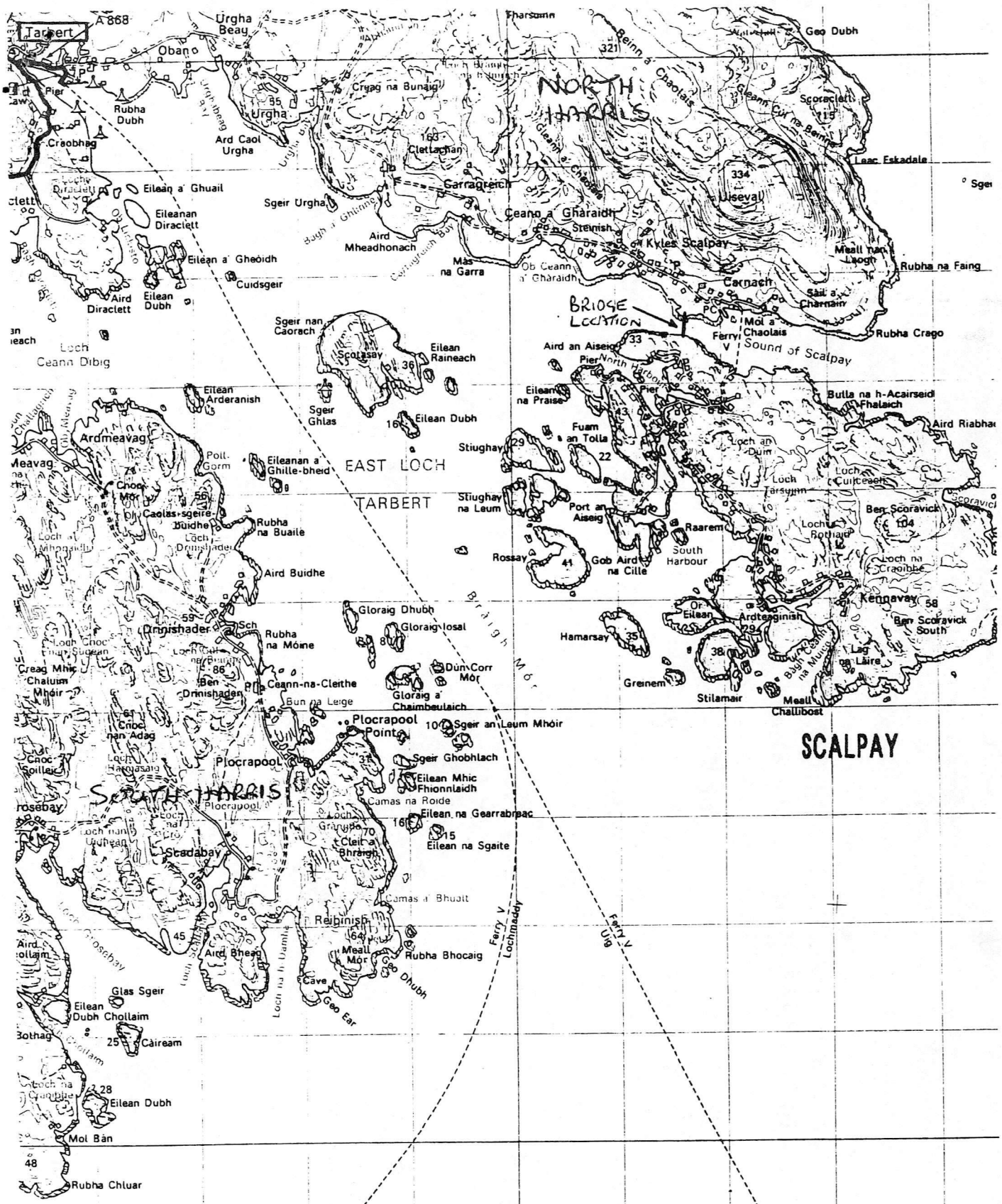

Figure 6.1 : Navigation approaches to Tarbert .

DRAWING REGISTER/ISSUE RECORD

PAGE No. 3

PROJECT TITLE: SCALPAY FIXED LINK

CLIENT: WESTERN ISLES ISLANDS COUNCIL

SHEET No. 2

JOB No. 12972

	DATE OF ISSUE					
DAY	30	27	3	17	9	12
MONTH	1	2	3	3	5	5
YEAR	95	95	95	95	95	95

DRAWING No.	DRAWING TITLE	REVISION					
12972 (04)/50	BRIDGE - GENERAL ARRANGEMENT	D	E	F			F
51	GENERAL ARRANGEMENT OF STEELWORK		/				
52	STEELWORK SQUANT LINES.		/				
53	GENERAL ARRANGEMENT OF PIERS.		/		A		
54	RING FRAME & DIAPHRAGM DETAILS.		/				
55	BOX GIRDER - TYPICAL DETAILS		/				
56	HAUNCH - GA & DETAILS			/			
57	SPLICE CONNECTIONS TYPE 1,2 & 3			/			
59	PILE BASE - GA & HOLDING DOWN DETAILS				/		
60	SUPERSTRUCTURE CONSTRUCTION SEQUENCE		B				
61	SCALPAY ABUTMENT - GENERAL ARRANGEMENT					/	
62	SCALPAY ABUTMENT - REINFORCEMENT DETAILS Sh 1 of 2					/	
63	SCALPAY ABUTMENT REINFORCEMENT DETAILS Sh 2 of 2						
64	SCALPAY SPRINGING - GENERAL ARRANGEMENT					/	

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KEY:

PURPOSE

1. PRELIMINARY
2. INFORMATION
3. APPROVAL
4. BILLING
5. TENDER
6. CONSTRUCTION
7. RECORD

STAGE

REC – RECORD
PT – PRE-TENDER
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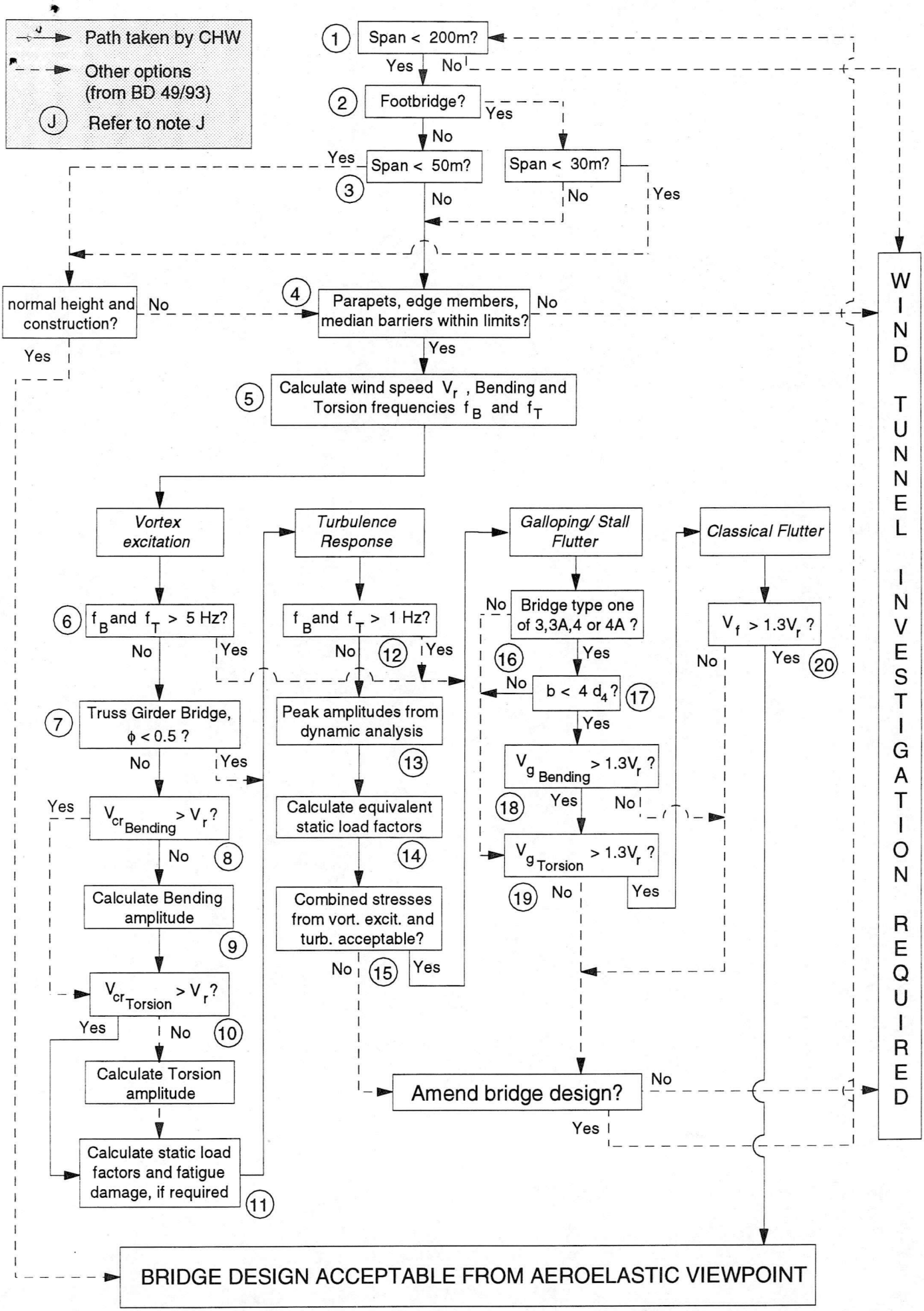

FIG. 1 Flow Chart of BD 49/93 code with route taken by CHW.