Technical Report Documentation Page

1. REPORT No.
CA-HY-ST-4173-41-1

2. GOVERNMENT ACCESSION No.

3. RECIPIENT'S CATALOG No.

4. TITLE AND SUBTITLE
Field Dynamic Studies Of A Long-Span Sign Structure

5. REPORT DATE
December, 1973

6. PERFORMING ORGANIZATION
14030

7. AUTHOR(S)
Henry D. Nix and Herbert Reinl

8. PERFORMING ORGANIZATION REPORT No.
CA-HY-ST-4173-74-1

9. PERFORMING ORGANIZATION NAME AND ADDRESS
State of California, Business and Transportation Agency,
Department of Transportation, Division of Highways Office of
Structures, Research and Development Branch

10. WORK UNIT No.

11. CONTRACT OR GRANT No.
D-4-131

12. SPONSORING AGENCY NAME AND ADDRESS
United States Department of Transportation
Federal Highway Administration

13. TYPE OF REPORT & PERIOD COVERED
Research 6/73- 12/73

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

16. ABSTRACT
A field study of a long-span, tubular signal bridge, very susceptible to wind-induced vibration, is described.

Portions of a theoretical analysis performed by a recently developed computer program for the evaluation of "Wind Effects on Luminaires and Traffic Signals" (WEFFLS), were verified by field testing. The predicated resonant frequencies and lowest critical wind velocity- that wind velocity which will cause structure resonance- were confirmed by the tests. An in-situ value of structure damping, 0.2% of critical damping, was measured.

17. KEYWORDS
Dynamics, fatigue, signals, structures, vibration, vortex shedding

18. No. OF PAGES:
58

19. DRI WEBSITE LINK
http://www.dot.ca.gov/hq/research/researchreports/1973/73-57.pdf

20. FILE NAME
73-57.pdf

5733

state of california

business & transportation agency

department of transportation

Caltrans

division of highways

office of structures

Field Dynamic Studies of a Long-Span Sign Structure

73-57

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration CA-HY-ST-4173-74-1

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle FIELD DYNAMIC STUDIES of a LONG-SPAN SIGN STRUCTURE		5. Report Date December, 1973 6. Performing Organization Code 14030
7. Author(s) Henry D. Nix, Associate Bridge Engineer Herbert Reinl, Associate Bridge Engineer		8. Performing Organization Report No. CA-HY-ST-4173-74-1
9. Performing Organization Name and Address State of California, Business and Transportation Agency, Department of Transportation, Division of Highways Office of Structures, Research and Development Branch		10. Work Unit No. 11. Contract or Grant No. D-4-131
12. Sponsoring Agency Name and Address United States Department of Transportation Federal Highway Administration		13. Type of Report and Period Covered Research 6/73 - 12/73 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

A field study of a long-span, tubular signal bridge, very susceptible to wind-induced vibration, is described.

Portions of a theoretical analysis performed by a recently developed computer program for the evaluation of "Wind Effects on Luminaires and Traffic Signals" (WEFFLS), were verified by field testing. The predicted resonant frequencies and lowest critical wind velocity - that wind velocity which will cause structure resonance - were confirmed by the tests. An in-situ value of structure damping, 0.2% of critical damping, was measured.

17. Key Words Dynamics, Fatigue, Signals, Structures, Vibration, Vortex Shedding		18. Distribution Statement Unlimited	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-69)

73-57

State of California

Business and Transportation Agency

Memorandum

~~JLBeaton~~ Library

To : All District Directors

Date: June 7, 1974

File : 700.57

From : DEPARTMENT OF TRANSPORTATION -Division of Highways
Office of Structures

Subject:

Attached is a copy of the final report on our research project entitled, "Field Dynamic Studies of a Long-Span Sign Structure," by H. D. Nix and H. Reinl. The report describes field measurements of structure natural frequencies, damping, and critical wind velocities.

The test structure is an example of a new type of sign/signal support structure intended to replace standard designs for box-beam and truss type structures for special situations. The new tubular designs are very susceptible to wind-induced vibration and related fatigue. A computer program developed in a preceding research project analyzes structures of this nature subjected to dynamic wind loads and develops an estimate of structure fatigue life. The tests described in the report verified certain theoretical foundations of the computer program.

The report notes that existing AASHTO criteria governing the design of sign support structures which have as their objective the elimination of vibration and fatigue, result in extremely uneconomical designs. No recommendations are made, however, to modify existing criteria since not all aspects of the fatigue life design method have been experimentally verified. It is anticipated that work will continue in this area.

Dale F. Downing

Dale F. Downing
Chief, Office of Structures

HDN:ahr

cc Copies of this memo have been sent to:

JLBeaton
WRGreen
TWSmith

June 7, 197[illegible]

700.57

-Division of Highways

[illegible] Structures

[illegible] a copy of the final report on our research project entitled, "Field Dynamic Studies of a Long-Span [illegible] Structure," by H. C. Wu and H. Reinl. The report [illegible] field measurements of structure natural frequencies, damping, and critical wind velocities.

[illegible] structure is an example of a new type of sign/signal [illegible] structure intended to replace standard designs for [illegible] cross type structures for special situations. [illegible] designs are very susceptible to wind-induced [illegible] related fatigue. A computer program developed [illegible] research project analyzes structures of this [illegible] subjected to dynamic wind loads and develops an [illegible] structure fatigue life. The tests described in [illegible] report verified certain theoretical foundations of the [illegible] program.

[illegible] report notes that existing AASHTO criteria governing the design [illegible] support structures which have as their objective [illegible] of vibration and fatigue, result in extremely [illegible] designs. No recommendations are made, however, [illegible] criteria since not all aspects of the [illegible] design method have been experimentally verified. [illegible] that work will continue in this area.

[illegible] Downing
[illegible], Bureau of Structures

Copies of this memo have been sent to:

[illegible]
[illegible]
[illegible]

Acknowledgments

The computer program discussed in this report, program WEFFLS, was developed by Mechanics Research, Inc., in conjunction with a research investigation initiated by the California Department of Transportation and financed in part, by the United States Department of Transportation, Federal Highway Administration.

The authors would like to thank Messrs. William Chow, Richard Johnson, and Del Gans of the California Department of Transportation, Division of Highways, Transportation Laboratory for their efforts in assembling and operating the test equipment.

Table of Contents

Page

List of Figures

Figure

List of Tables

Foreword

There has been increasing interest in the deleterious effects of wind-induced vibration of luminaires and sign/signal bridges in recent years. California has shared this interest and is actively working to develop more rational design criteria for structures susceptible to aeolian vibration. An outgrowth of California's efforts has been the development of a computer program for the evaluation of "Wind Effects on Luminaires and Traffic Signals" (WEFFLS).

Program WEFFLS attempts to evaluate the static, dynamic, and fatigue adequacy of a particular class of structures. This research investigation was undertaken to provide an experimental verification of certain fundamental aspects of the WEFFLS analysis. The project cannot be construed as a complete experimental verification of program WEFFLS. WEFFLS is, and will remain for an indefinite period, an experimental and developmental tool, and should be viewed only in such a context.

1. Introduction

The increasing use of tubular metal members in traffic signal and luminaire supports has forced a consideration of wind-induced vibration in the structural design. In California, several luminaire failures prompted an investigation into the cause of failure and a subsequent review of design criteria.

The investigation determined the cause of failure to be fatigue cracking of welds which joined the structure's base plate to the vertical pole. The design review revealed that few tools were available for the structural design of tubular metal members which considered metal fatigue arising from wind-induced vibration. A research project (4)* was initiated to develop an analytical method which would consider fatigue criteria in the design of typical standards. The project terminated with the development of a computer program which has the acronym WEFFLS (Wind Effects on Luminaires and Traffic Signals).

The analysis performed by program WEFFLS includes consideration of fatigue arising from wind gusts and steady-state vibration developed by vortex shedding. The tested signal structure would vibrate quite visibly in the first in-plane mode at low wind velocities. This fact suggested the occurrence of vortex shedding and that, for this particular structure, the fatigue component arising from vortex shedding would be much more significant than the fatigue developed by wind gusts.

(*Numbers in parentheses refer to references in the Bibliography.)

The experimental program described in the following pages was intended to verify certain fundamental aspects of the WEFFLS analysis. The experimental emphasis was on vortex shedding related parameters since the tested structure was particularly susceptible to this phenomenon. An experimental program to verify completely all aspects of the theoretical analysis would be very long and complex, although certainly warranted.

2. General Description of Vortex Shedding

Whenever a blunt object is immersed in a moving fluid, the phenomena of vortex shedding occur. A region of turbulence develops on the object, which grows larger in time, until a portion of the turbulence is shed in the form of a vortex. The vortex travels around the body to some point at which it separates from the body surface and joins the fluid stream. The vortices are shed on alternate sides of the body at a particular rate, the vortex shedding frequency.

The vortices travel at a velocity somewhat less than the fluid on the opposing side of the member. Thus, by Bernoulli's Law, a pressure differential exists between the opposite sides. The pressure difference may be viewed as an alternating force that acts at right angles to the direction of fluid flow on the structure.

Experiments have shown the frequency of vortex shedding to be a function of fluid velocity, a characteristic dimension, and a shape coefficient, the Strouhal Number. The equation expressing the relationship is the following:

$$f_v = \frac{SV}{d}$$

where f_v = frequency of shedding vortex pairs

S = Strouhal Number

V = fluid velocity

d = characteristic dimension

For a circular member, d is usually taken as the outside diameter and S is approximately 0.19 for a wide range of Reynold's numbers.

With respect to sign and supporting structures, vortex shedding forces become significant if the ambient wind velocity and structure dimensions combine to produce vortices at a frequency close to a structure's natural frequency. In such a situation resonance may produce significant, varying member stresses. These stresses are limited only by the wind duration and structure damping.

An assessment of fatigue, which might develop from vortex shedding induced steady-state vibration, must be founded on a knowledge of the structure's natural frequencies, damping, and wind environment. All three are equally important parameters.

3. Objectives of Test Program

The test program had three primary objectives:

1. To verify the WEFFLS-predicted natural vibration frequencies;

2. To assess the structure damping ratio; and
3. To determine experimentally the lowest wind velocity which causes resonance in the test structure.

The prototype structure, a long span, tubular, continuous traffic signal bridge, was tested with and without traffic signals in place.

4. Description of Test Structure

The test structure is located at the intersection of State Routes 238, 185, and 92 in the City of Hayward, California. The structure spans 112 feet across the intersection and is constructed of tubular galvanized steel pipe of 10-3/4-inch outside diameter. The wall thickness is .365 inches on the vertical support and 0.188 inches on the horizontal member. The tubular section is of constant outside diameter.

The structure is continuous and rests on cast-in-drilled-hole piles twelve to eighteen feet deep. The connection of the structure base plate to the pile foundation was accomplished by means of four two-inch diameter bolts. Figure 1 depicts the structure with pertinent dimensions.

5. Description of Instrumentation

The structure was tested on three occasions. The first testing was intended to check the test equipment and procedures. The second testing involved more measurements than the first, including the recording of ambient winds. The first and second testings were performed on the bare frame. In the third phase the same equipment and procedures were utilized to test the frame with signals attached. The three test occasions will henceforth be labeled as Tests A, B, and C.

5.1 Test A

The structure was excited by an Electro-Seis shaker, placed such that the structure was induced to vibrate in its own plane. The shaker transmitted a sinusoidally varying horizontal force to the frame at a location five feet above one support.

The horizontal components of the structure's accelerations and displacements induced by the shaker were measured at a location 7.5 feet above the support by a Statham Accelerometer, A4-1.5-350, with a range of ±1.5g and a BLH Displacement Transducer with a range of ±.35 inches, respectively. The Test A instrumentation is portrayed in Figure 2.

5.2 Test B

The Electro-Seis shaker was again used to impart an alternating force to the structure. The shaker was positioned ten feet above one structure support rather than five feet, as in Test A. The change in position of the shaker was simply for experimental convenience and was not a consequence of any Test A findings.

Accelerations and displacements were recorded at two structure locations, the midspan and one quarter span point of the horizontal member. The vertical component of motion was measured since the in-plane structure motion is primarily vertical at the instrumentation locations. The accelerations were measured with Statham Accelerometers.

An optical measuring system, the Optron, was used in tests B and C to determine the midspan and quarterspan deflections. The Optron system is composed of a black and white target and a sensing unit. The sensing unit can be remote from the target and will track target movements which occur normal to the axis formed by the target and sensor. There are no physical connections between target and sensor, but there must exist an unrestricted light path from target to sensor.

The lack of physical connections between target and sensor was very advantageous. The tested structure spans a busy urban intersection. The traffic volume at that location precluded the use of any instrumentation which would require closing a traffic lane or would otherwise interfere with traffic flow. The Optron system was well suited to the requirements of this project.

In addition to the acceleration and displacement measurements, wind speed and direction were also recorded. The wind speed was recorded at 14 and 31 feet above the base of the signal standard. The wind direction was recorded only at the higher level. The location of the test apparatus for Test B is presented in Figure 3.

5.3 Test C

The instrumentation layout and testing procedures were similar to those detailed for Test B, except that no measurements of wind speed and direction were made for reasons outlined in the following section. Tests A and B were made with the structure in its "bare frame" condition; that is, there were no signal heads or signs affixed. For Test C, however, the structure was in its final configuration with nine programmed visibility head signals in place. The structure in its final configuration is shown in Figure 1.

6. Description of Test Procedure

A primary test objective was to verify the computer predicted natural frequencies. A straightforward method of determining the structure's natural frequencies was chosen in order to minimize post-testing data reduction. In Test A, comparisons were made between the input signal to the Electro-Seis shaker and the output signal from the accelerometer and deflectometer. The accelerometer, Optron, and input signals were compared in Tests B and C.

The comparisons of input with output could be made visually since all transducer output, including the driving force, were recorded on an oscillograph. It was possible to sweep a frequency range with a variable oscillator, which provided a sinusoidal input signal to the shaker, and note when similar response curves were obtained from the deflectometer (or Optrons) and accelerometer. At resonance the curves appear in phase and the frequency of the driving signal represents a natural structure vibration frequency.

With the computer predictions as a guide, the oscillator could be tuned to the structure's natural frequencies. In fact, due to the high flexibility of the structure, the lower modes were clearly visible to the eye. The frequency spectrum was scanned from about 1 H_z to 16 H_z which encompassed the structure's lowest twelve natural frequencies. Since only the first twelve modes are included in the WEFFLS analysis, the testing was restricted to that range of frequencies.

The structural damping was determined from a displacement amplitude curve. The structure was excited in a particular mode to a steady-state condition and the driving force was then eliminated. The damped free vibration decay curve was recorded on the oscillograph. Decay curves were obtained for one lower mode and one higher mode. Appendix A details the method by which damping ratios were calculated and presents the experimental values.

The third test objective was to determine the lowest wind speed which caused resonance in the structure. It was possible to record simultaneously the accelerations, displacements, wind speed and direction at a time when the structure was visibly vibrating. The oscillograph traces of acceleration and displacement indicated that the structure was vibrating at one of the previously determined natural frequencies. An analysis of the recorded wind speed and direction revealed the critical wind velocity. This procedure was performed only for the structure without signal heads installed

At the inception of the test program, it was planned that ambient wind measurements would be made for both Tests B and C; however, after the signal heads were installed, the structure did not respond to low velocity winds as it did as an uncluttered, bare frame. At no time during the Test C measurements was the structure observed resonating under the influence of ambient winds. Apparently, the addition of the signal heads disrupted the air flow to the extent that significant vortex shedding forces could not be generated. Had ambient wind measurements been made, their only value would be to define a range of wind speeds in which the critical wind velocity was not reached. Test C ambient winds were felt to be about the same magnitude and direction as in Test B.

7. Test Results

7.1 Natural Frequencies

The natural frequencies determined during Tests A, B, C are presented in Table 1. The table includes a list of the first twelve WEFFLS-predicted natural frequencies. The notations "out" or "in" signify a primarily out-of-plane or in-plane vibration, respectively. The test objective was to determine the in-plane frequencies only, since it is the corresponding in-plane modes that will be activated by the vortex shedding forces.

There is good agreement between the measured frequencies in

Tests A and B. This is to be expected since the structure was in a similar configuration for both tests. Measurements of the natural frequencies were repeated in Test B since that test phase had more instrumentation than that which was used in the initial phase, Test A. The reasonable agreement between measured frequencies in Test A and Test B suggests that the simpler test procedure, Test A, would be adequate for future studies.

The measured resonant frequencies for the structure with signals installed, Test C, are lower than those determined without signal heads installed, Test B, for the first three in-plane modes. The addition of nine sixty-six pound signal heads caused the drop in frequencies. This point will be further elaborated after a discussion of the theoretical results.

It is interesting that, for the structure with and without signal heads in place, the measured natural frequencies were lower than the predicted. This fact suggests some support flexibility, the importance of which will be considered in the theoretical analysis discussion.

7.2 Structure Damping

The structure damping was determined in two modes - mode 6 at 4.79 Hz and mode 3 at 2.20 Hz. A critical damping ratio in mode 3, as measured over 100 cycles, of 0.0024 was obtained. The corresponding number for mode 6 was 0.0019.

A reasonable average would be 0.2% critical damping. (See Appendix A for calculation details.)

7.3 Critical Wind Velocity

When it was visually obvious that the structure was vibrating under the influence of vortex shedding forces, recordings were made of wind velocity and direction and structure acceleration and displacement at the quarter-span and mid-span points. The critical frequency was determined from the acceleration trace. The wind velocity normal to the plane of the structure was calculated from the recorded velocity and direction. The critical wind velocity was found to lie between 6 and 9 miles per hour. However, difficulty encountered in aligning the axis of the directional vane with the plane of the structure could have introduced an orientation error of up to 10°. If an alignment error of 10° is assumed, the measured critical wind velocity range is from 5 to 8 miles per hour.

A brief description of the wind tower is necessary to understand how a seemingly large alignment error could be introduced in the directional vane. The anemometer and directional vane were mounted on a special wind tower manufactured by the Division of Highways, Transportation Laboratory for air pollution studies. The tower is composed of telescoping sections which utilize the transporting trailer as a base. When being transported, the tower is telescoped and rests in a horizontal position on the trailer. The anemometer and directional vane, which must be oriented to a reference direction, are positioned while the tower is horizontal. The tower is rotated to the vertical by a winch and cable and is extended by a second winch and cable system. The orientation of the transporting trailer and the 90° tower rotation during erection contribute to the difficulty in aligning the directional vane.

8. Theoretical Results

Of the three test objectives outlined early in the report, two were to verify experimentally theoretically determined values. These values were the theoretical predictions for the structure's natural frequencies and the lowest critical wind velocity. (A critical wind velocity is defined as that wind speed which, when acting normal to the plane of the structure, will cause structure vibrations at a natural frequency through the mechanism of vortex shedding.) The natural frequencies and critical wind velocities are fundamental to the problem of wind-induced vibration of linear systems. The theoretical determination of each of these quantities will be reviewed in the following pages.

8.1 Natural Frequencies

Program WEFFLS formulates the structural problem by the direct stiffness method. The natural frequencies and mode shapes (the classical eigenvalue - vector problem) are determined by Householder Tridiagonalization with preliminary Cholesky Decomposition of the dynamical matrix. A lumped mass model is used to describe the structure for dynamic purposes. Rotational inertia is neglected. The mathematical formulation using these techniques and assumptions is well documented in the literature and will not be presented herein.

The adequacy of any theoretical predictions of natural frequencies and mode shapes is dependent on the fineness of the structural discretization used, the assumptions built into the solution process, and the boundary conditions. The structural discretization used in the WEFFLS analysis is shown in Figure 4. Model 1 included nineteen nodes and eighteen members. The WEFFLS-predicted natural frequencies and mode shapes were checked by analyzing the structure with an identical discretization using the STRUDL (Structural Design Language) subsystem of ICES (Integrated Civil Engineering System) (2,3). Because WEFFLS was programmed for a maximum of 20 nodal points, the structure was then further subdivided and reanalyzed using STRUDL. The objective was not to validate the mode determination as done by either WEFFLS or STRUDL, but rather to establish the precision with which the original structural discretization (which had been cross-checked between the two programs) captured the natural frequencies and mode shapes. The second structural discretization, Model 2, used 23 nodes and 22 members. In all cases, the structure was assumed to be fully restrained against rotation and displacement at the base nodes.

Table 2 contains the results of the single WEFFLS and the two STRUDL analyses. There is very little difference between the two programs for structural Model 1 and little difference between the results for Models 1 and 2 as analyzed by STRUDL except in the eleventh and twelfth modes.

The first six mode shapes for structural Model 1 as determined by WEFFLS and STRUDL are presented in Figures 5 and 6. Each mode shape is plotted as a single line since there is negligible difference between the program predictions. The first six modes include three in-plane and three out-of-plane vibrations.

The structure was reanalyzed as structural Model 3 with STRUDL to determine the natural frequencies with the nine signal heads in place. STRUDL was used rather than WEFFLS for this modelling because of the 20 node limitation in WEFFLS. A very representative model was developed with each signal head defining a nodal point. An accurate mathematical model was desired since comparisons were to be made between theoretical and experimental values for natural frequencies. A coarse mathematical model would have introduced an apparent disagreement.

It should be noted, however, that a 20 node WEFFLS model would be adequate for the ultimate goal of a fatigue analysis. For example, a measurement may yield a critical wind velocity of five mph and a theoretical prediction, four mph. The theoretical value is apparently in error by 20%. The probability of the occurence of a four mph wind is nearly the same as that for a five mph wind and the number of cycles a given structure would be subjected to at each wind level are approximately equal. Thus, when the probabilistic fatigue analysis is done, the consequences of seemingly large errors in critical wind velocities are lessened.

In the preceding analyses, the structure was modeled with full fixity at the supports. The cast-in-drilled hole pile foundation is quite rigid; however, the bolted base plate connection does have some flexibility. A theoretical or experimental determination of this flexibility would be difficult. Consequently, the natural frequencies were determined by additional theoretical analyses with the structure permitted to rotate in its plane at the base nodal points. The out-of-plane and torsional rotational restraints were retained. The combination of analyses with and without in-plane rotation provided a range of theoretical natural frequencies within which the measured frequencies should fall. The results of these analyses are listed in Table 3 and will be discussed more fully in a later section of the report.

8.2 Critical Wind Velocities

The equation presented earlier in the report.

$$f_v = \frac{SV}{d},$$

can be written as

$$V_c = \frac{fd}{S}$$

to yield wind velocity as a function of frequency, member diameter, and Strouhal number. The critical wind velocities are those wind speeds which will cause vortices to be shed at a frequency that corresponds to a structure natural frequency.

Critical wind velocities were calculated for the natural frequencies corresponding to the structure without signals and for the fixed/pinned base conditions. The calculations were made only for the in-plane modes through mode 10, with $S = 0.19$, $d = 10.75$. The results of the calculations appear in Table 4.

9. Comparison of Theoretical with Experimental Results

9.1 Natural Frequencies

Table 3 presents the theoretical natural frequencies for the boundary conditions of fixed and released in-plane rotation at the base. Through mode 6, the theoretical frequencies provide bounds for the experimentally determined natural frequencies. The experimental frequencies are closer to the theoretical fixed end frequencies in the lower modes, for the structure with and without signal heads installed, than to the pinned end frequencies. Thus, the fully fixed theoretical model used in the WEFFLS analysis is the better approximation.

Table 3 also demonstrates the anticipated drop in frequency due to the addition of the signal heads to the structure in all measured modes but number 8. The frequency drop is a simple consequence of the addition of mass to a vibrating system while retaining the same value of stiffness.

The experimentally determined natural frequencies for modes 8 and 10 fall outside the theoretical range. This anomaly may be due to a poor determination of the higher frequencies. An exact description of the higher frequencies is not necessary, however, since damage due to vortex shedding is confined to the lower modes for which higher stresses are developed.

In general, the experimentally determined natural frequencies agree quite well with the theoretical predictions. The use of a fully fixed boundary condition for analysis purposes appears justified for the cast-in-drilled-hole pile foundation detail.

9.2 Structure Damping

Since the test structure has only bolted splices and two bolted base connections, joint flexibility probably contributes very little to energy dissipation. The major damping mechanism is most likely viscous damping from the surrounding air. The WEFFLS analysis is formulated assuming viscous damping. Consequently, the experimental free vibration curve was reduced on the basis of viscous damping to obtain a critical damping ratio of 0.2%.

The critical damping ratio was obtained for two in-plane modes with similar results. Thus, the damping may be assumed mode independent for theoretical analyses. A fatigue analysis would be particularly susceptible to damping ratio since low damping would result in higher displacement amplitudes with corresponding higher stresses. Higher stresses would increase fatigue damage, assuming a cumulative damage model as used in program WEFFLS.

9.3 Critical Wind Velocity

The theoretical bounds for the structure's natural frequency in mode 2, the first in-plane mode, yield a range of from 3.6 to 5.9 miles per hour for critical wind velocity. The experimental mode 2 frequency of 1.67 H_z yields a critical wind velocity of 5.4 miles per hour. From the recordings of wind speed and direction made when the structure was vibrating in mode 2 under the influence of the wind alone, a velocity range of from 6 to 9 mph was obtained. Considering the possible alignment errors, previously discussed, there is adequate agreement between the theoretical critical wind velocity and the experimental values.

Experimental critical wind velocities for higher modes were not measured since the wind source was natural and, therefore, uncontrollable. The structure was not observed vibrating in any but the first in-plane mode under the influence of ambient winds during any test. Even though the higher modes have a theoretical existence, they may not be realized in nature on structures of this type due to the complex pattern of forces required to initiate and sustain higher mode vibration. If this conjecture proves to be true, the problem of wind-induced fatigue may be greatly simplified.

10. Design Considerations

The experimental work described herein was intended to provide verification of a developing design method. It would be worthwhile to examine the current American Association of State Highway Officials (AASHO) methods of handling vortex shedding in contrast with an alternative treatment of the encompassing fatigue problem.

Existing AASHO Specifications (7) attempt to eliminate resonance at critical wind speeds by a deflection control and, in the case of aluminum structures, with the additional use of damping or energy absorbing devices. In the case of steel structures, where auxiliary damping devices are not required, the deflection control places a significant penalty on the use of shapes similar to the tested structure--long span, thin-walled tubes. This observation has been made by others, notably Pelkey in his discussion of similar tubular sign support structures (6).

The deflection control criterion is developed by noting that the fundamental frequency of vibration of a simply supported beam of uniform mass and stiffness is approximately the reciprocal of the square root of the dead load deflection. That is,

$$f_0 \doteq \Delta^{-1/2}$$

where f_0 = fundamental frequency of vibration

Δ = dead load deflection

The frequency, f_v, at which objects shed vortex pairs is given by

$$f_v = \frac{SV}{d}$$

where S = Strouhal number

V = wind velocity

d = characteristic object dimension

The $d^2/400$ criterion, a limiting value for dead load deflection, is obtained when the structure's fundamental frequency, expressed as a function of dead load deflection, is equated to the vortex shedding frequency for a wind velocity of 80 mph.

For long spans, say greater than 60 feet, the $d^2/400$ criterion imposes an increasingly severe penalty in added material. An adequate static design will need an increase in pipe diameter and/or wall thickness where thin-walled pipe structural shape is proposed, to meet the deflection criterion.

The thin-walled pipe is a poor structural configuration from the standpoint of efficiency of material use since increases in stiffness must be obtained by increases in pipe diameter and/or wall thickness. The unit increase in stiffness per unit of material is much less than for other structural configurations, say I-beams or trusses. The cylindrical, thin-walled shape does have significant advantages, however, which tend to outweigh its structural disadvantages. The advantages include an uncluttered, pleasing appearance, straightforward fabrication, ease of erection, low painting costs due to the simple surface, and low drag. These advantages may not be sufficient to justify economically thin-walled pipe designs if the deflection criterion is retained.

For example, a value of 1.46 Hz was measured for the first in-plane natural frequency of the tested structure with signals in place. The traffic signal bridge has an outside pipe diameter of 10.75 inches. These values combine to predict a resonant wind velocity of 4.69 mph.

The critical wind velocity is a function of the square of the pipe diameter for tubular shapes. That diameter, D_2, which would satisfy the AASHO criterion of no resonance below 80 mph is given by the following equation:

$$D_2^2 = \frac{80 \times S \times 17.6 \times D_1}{f_{D_1}}$$

where D_2 = diameter at 80 mph

S = Strouhal number (.19)

D_1 = diameter at frequency

f_{D_1} = frequency, Hz

For values, D_1 = 10.75 inches, f_{D_1} = 1.46 Hz, D_2 becomes 44.4 in. An increase in pipe diameter of 44.4/10.75 or over four times would be required to satisfy the AASHO deflection criterion.

Clearly, the aesthetically pleasing tubular designs cannot be competitive if the existing AASHO deflection criterion is retained. An alternative design method is required to accommodate tubular structural shapes.

A second approach to the design of structures in which vortex shedding forces are anticipated is to consider the fatigue produced by the force and not attempt to eliminate resonance entirely.

The first condition imposed by this approach is to insure that stresses developed during resonant vibration are within allowable limits. Since the stress amplitudes are very sensitive to structural damping, it is important that an accurate figure be used. Comparative analyses performed by the authors with the 5% critical damping, commonly assumed for design, and a more realistic 1% critical damping, which is in keeping with the value obtained in these tests, reinforce this observation.

If the structure is permitted resonant vibration, then a large number of stress reversal cycles can be accumulated in a short time. Assuming the first critical wind velocity for the tested structure, 4.7 mph, could be sustained for a 24-hour period, nearly 30,000 stress reversals would occur. It is, however, very unlikely that the critical wind velocity would be maintained for such a period of time. The wind velocity distribution in time, considering only the component which will activate the vortex shedding phenomena, becomes of paramount importance when evaluating stress reversals and related fatigue.

A probabilistic model of the wind magnitude and frequency can be employed to estimate fatigue damage arising from the ramdom variation of wind gusts, the loading of the structure under extreme wind conditions, and the vibration of the structure due to vortex shedding forces. A tentative design which will permit resonant vibration can be evaluated with respect to fatigue life. Design revisions can then be made on a rational basis. California is currently developing such a design method.

11. Research Recommendations

The sensitivity of the vibration amplitude to critical damping ratio requires that an accurate figure be used for analysis. The value obtained for the test structure, 0.2%, was quite low due partially to the fact that signal heads had not yet been installed when the structure was tested for damping. Tests should be performed to evaluate damping for sign/signal support structures of different structural configuration and under service conditions.

Verification of the fatigue life predicted by the fatigue design method is needed. The tests should be constructed to incorporate the wind environment existing for in-service structures. These tests would provide a final substantiation of both the fatigue design method and the data on which the method is based.

Further work is needed to develop or assess existing vibration dampers for use on structures such as described in this report. Any rational basis on which the AASHO deflection criterion could be relaxed would generate considerable savings. For example, vibration dampers could be used to reduce structure vibrations to an acceptable level. It would, then, not be necessary to raise the structure's natural frequencies, to avoid harmful vibration, by increasing material dimensions.

12. Summary of Observations and Conclusions

The test program had three primary objectives:

1. To verify the theoretically predicted natural vibration frequencies;

2. To assess the structure damping ratio; and

3. To determine experimentally the lowest wind velocity which causes resonance in the test structure.

Good verification of theoretically predicted quantities was obtained for natural frequencies and resonant wind velocities. Successful measurements were made of the structure damping.

The measured in-plane natural frequencies fell within the theoretical bounds through mode 7. The experimental values agreed closely with the rigid boundary condition theoretical model.

The structure damping, assuming a viscous damping model, was measured in two modes. The critical damping ratio for both modes was about 0.2%. The similarity between the measured values in modes 3 and 6 suggests that the damping is mode independent.

A wind velocity of 5 to 8 miles per hour was observed to activate the second structure mode (the first in-plane mode) without signals in place. The measured second mode frequency yields a critical wind velocity of 5.4 mph. There is satisfactory agreement between the measured and predicted wind velocities in light of the complexities of the vortex shedding problem.

13. Summary Statement of Implementation

A theoretical approach to the problem of accumulated fatigue in structures subjected to wind was developed as part of a project entitled, "Aerodynamically Induced Fatigue Stresses in Traffic Signals and Luminaire Supports" (4). This project was financed, in part, by the United States Department of Transportation, Federal Highway Administration. A computer program, WEFFLS, was written to evaluate numerically the fatigue life of selected structural types.

The work performed in connection with this research investigation is a partial verification of the theoretical basis of the program, WEFFLS. This program is currently being used as an aid in the development of new standard designs. The new designs consider fatigue as a controllable quantity. Structural parameters are adjusted to yield a predicted fatigue life commensurate with the design structure life.

The verifications provided by this investigation are an integral part of a developing design method. As refinement of the design technique continues, further experimental work would be justified to verify the remaining theoretical foundations.

Bibliography

1. Lengel, J. S., Sharp, M. L., "Vibration and Damping of Aluminum Overhead Sign Structures", Highway Research Record, No. 259, 1969.

2. Logcher, R. D., Flashbart, B. B., Hall, E. J., Power, C. M., Wells, R. A., and Ferrante, A. J., ICES STRUDL-II Engineering User's Manual, Volume 1 Frame Analysis, Massachusetts Institute of Technology, School of Engineering, Research Report R68-91, November 1968.

3. Logcher, R. D., Connor, J. J., and Nelson, M. F., ICES STRUDL-II Engineering User's Manual, Volume 2 Additional Design and Analysis Facilities, Massachusetts Institute of Technology, School of Engineering, Research Report R70-77, June 1971.

4. Haelsig, R. T., et. al, "Aerodynamically Induced Stresses in Traffic Signals and Luminaires Supports", Mechanics Research, Inc., Tacoma, Washington, 1971.

5. Hurty, W. C., Rubinstein, M. R. "Dynamics of Structures", Prentice-Hall, Inc., Englewood Cliffs, N. J., 1964.

6. Pelkey, R. E., "New Design Approach to Long-Span Overhead Sign Structures", Highway Research Record, No. 346. 1971.

7. "Specifications for the Design and Construction of Structural Supports for Highway Signs", American Association of State Highway Officials, 1968.

8. "Specifications for the Design and Construction of Structural Supports for Highway Luminaires", American Association of State Highway Officials, 1971.

TRAFFIC SIGNAL BRIDGE

Table I

STRUCTURE NATURAL FREQUENCIES

THEORETICAL (WEFFLS)			EXPERIMENTAL		
			Without Signals		With Signals
Mode		Frequency Hz	Test A Frequency, Hz	Test B Frequency, Hz	Test C Frequency, Hz
1	OUT	1.12	—	—	—
2	IN	1.84	1.68	1.67	1.46
3	IN	2.52	2.20	Not Measured	2.02
4	OUT	2.64	—	—	—
5	OUT	3.98	—	—	—
6	IN	5.60	4.82	4.72	4.41
7	OUT	6.24	—	—	—
8	IN	9.11	7.55	7.56	7.58
9	OUT	10.03	—	—	—
10	IN	14.04	11.8	12.0	11.5
11	IN	15.44	Not Measured	Not Measured	Not Measured
12	OUT	16.00	—	—	—

Mode	THEORETICAL NATURAL FREQUENCY Hz		
	WEFFLS	STRUDL	
	Model 1	Model 1	Model 2
1	1.12	1.12	1.11
2	1.84	1.81	1.83
3	2.52	2.47	2.46
4	2.64	2.60	2.58
5	3.98	3.88	3.98
6	5.60	5.51	5.54
7	6.23	6.15	6.33
8	9.11	9.05	8.75
9	10.03	9.97	9.84
10	14.04	13.63	13.58
11	15.44	15.11	14.68
12	16.00	15.44	18.88

TRAFFIC SIGNAL BRIDGE

Table 2

COMPARISON OF THEORETICAL NATURAL FREQUENCIES

TRAFFIC SIGNAL STRUCTURE

Table 3

NATURAL FREQUENCIES FOR FIXED/PINNED BASE CONDITIONS

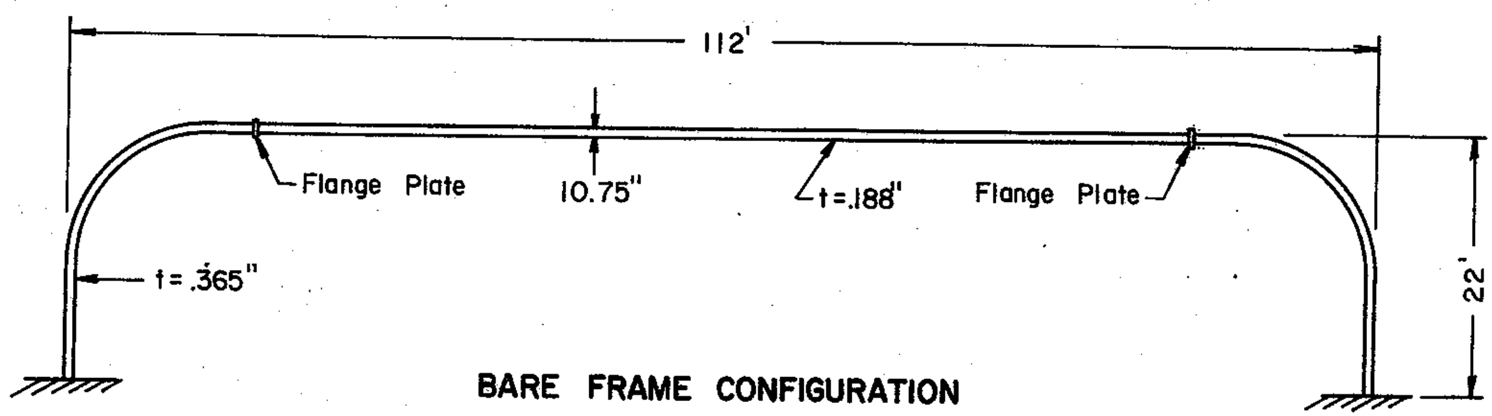
Mode	Without Signals			With Signals		
	Theoretical Fixed (STRUDL)	Experimental	Theoretical Free (STRUDL)	Theoretical Fixed (STRUDL)	Experimental	Theoretical Free (STRUDL)
1	1.11		1.11	.94		.94
2	1.83	1.68	1.11	1.51	1.46	1.02
3	2.46	2.20	1.72	2.24	2.02	1.42
4	2.58		2.58	2.42		2.42
5	3.98		3.98	3.75		3.75
6	5.54	4.82	4.56	4.92	4.41	4.09
7	6.33		6.33	5.86		5.86
8	8.75	7.55	7.96	8.03	7.58	7.34
9	9.84		9.84	9.19		9.19
10	13.58	11.8	12.16	12.35	11.5	11.08
11	14.68		14.68	13.48		13.48
12	18.88		16.83	17.16		15.42

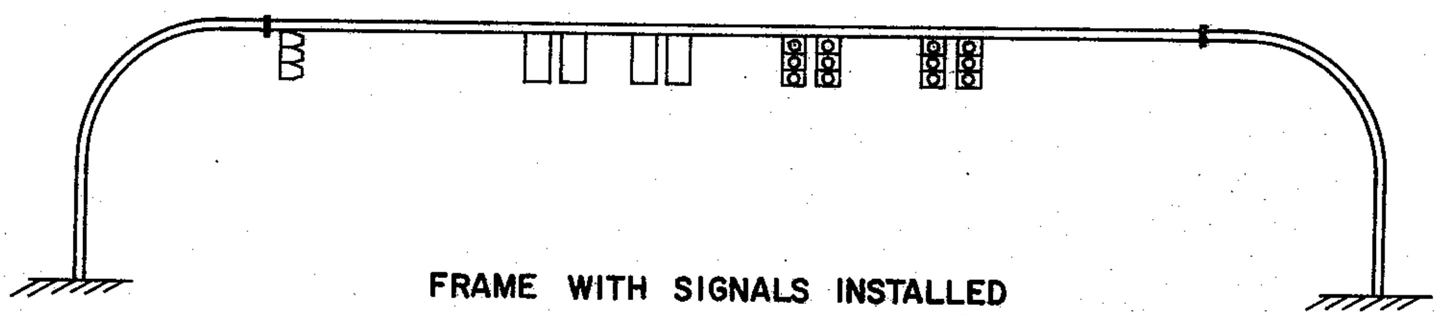
TRAFFIC SIGNAL STRUCTURE

Table 4

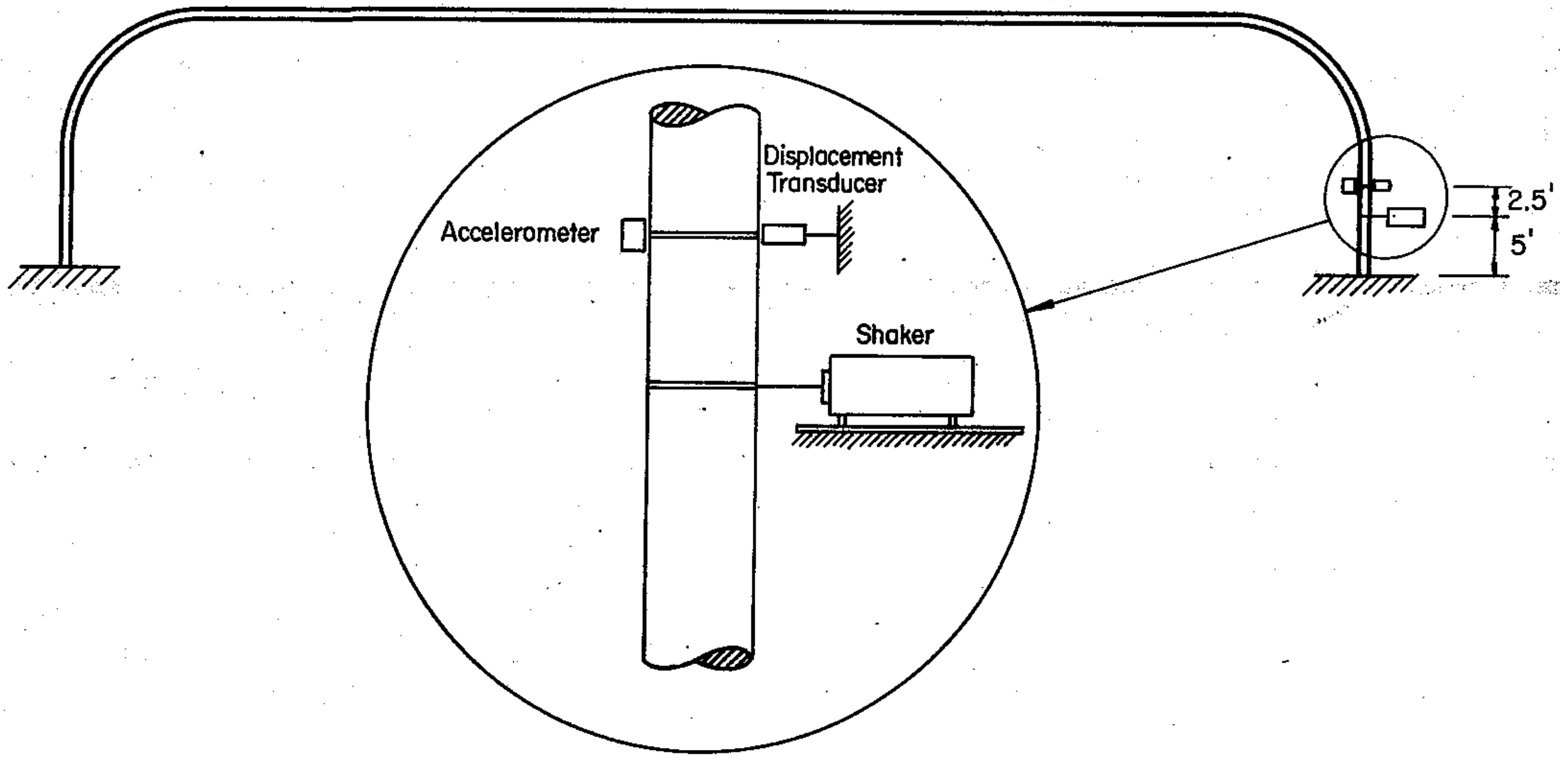
CRITICAL WIND VELOCITIES

Mode	End Condition			
	Fixed		Free	
	Frequency Hz	Critical Wind Speed, mph	Frequency Hz	Critical Wind Speed, mph
2	1.83	5.9	1.11	3.6
3	2.46	7.9	1.72	5.5
6	5.54	17.8	4.56	14.7
8	8.75	28.1	7.96	25.6
10	13.58	43.6	12.16	39.1

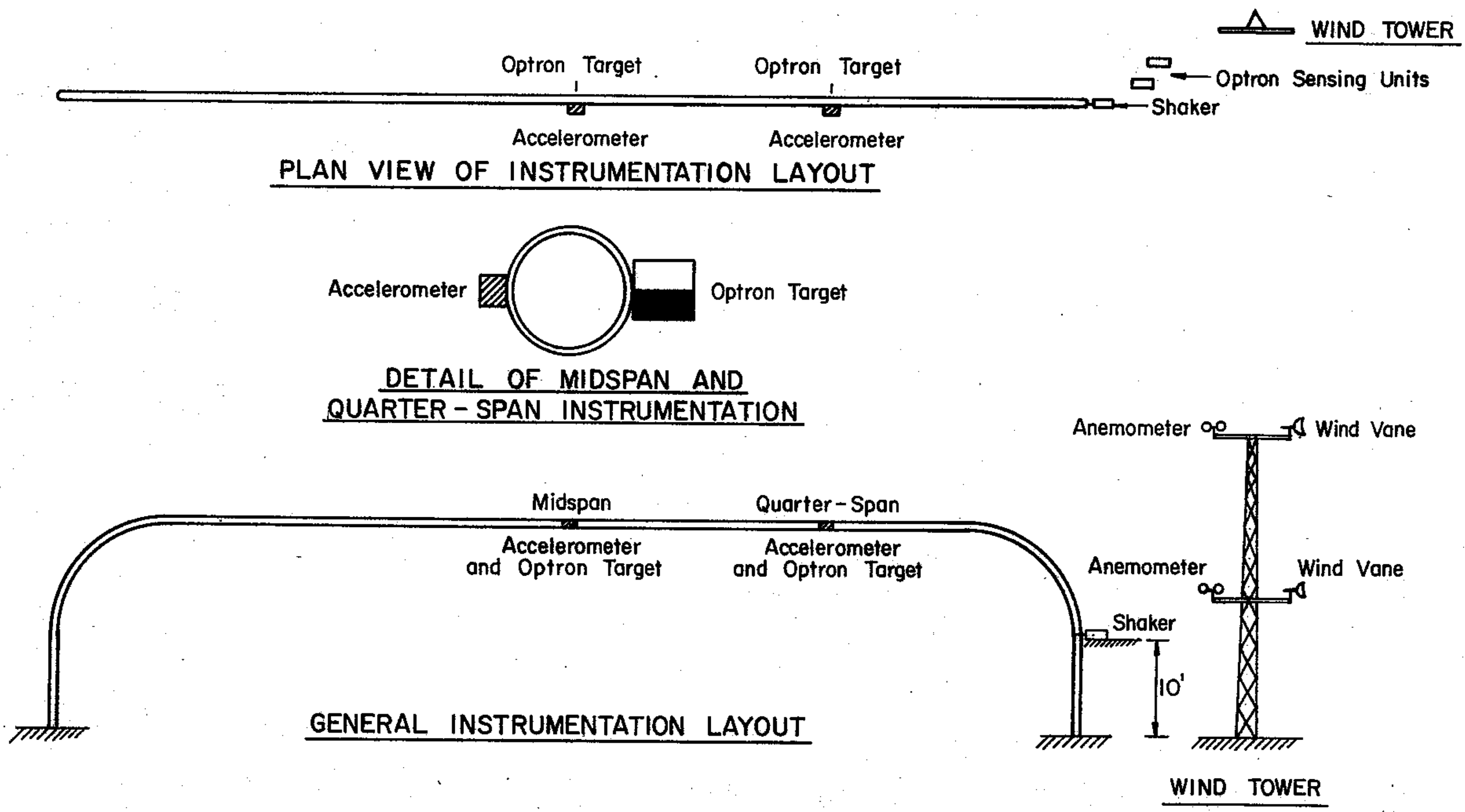




TRAFFIC SIGNAL BRIDGE
Figure 1
STRUCTURE DIMENSIONS



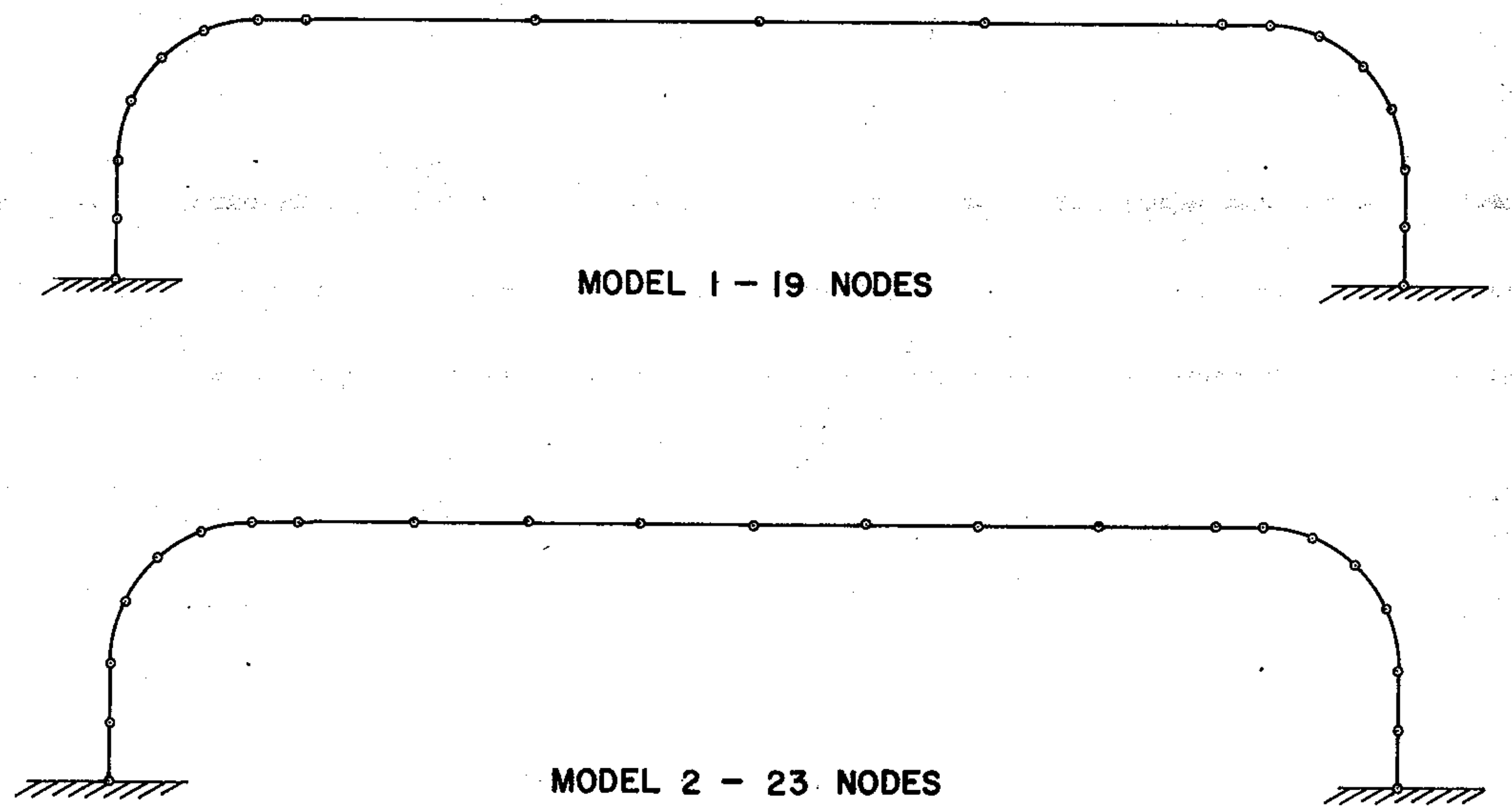
TRAFFIC SIGNAL BRIDGE
Figure 2
TEST A INSTRUMENTATION



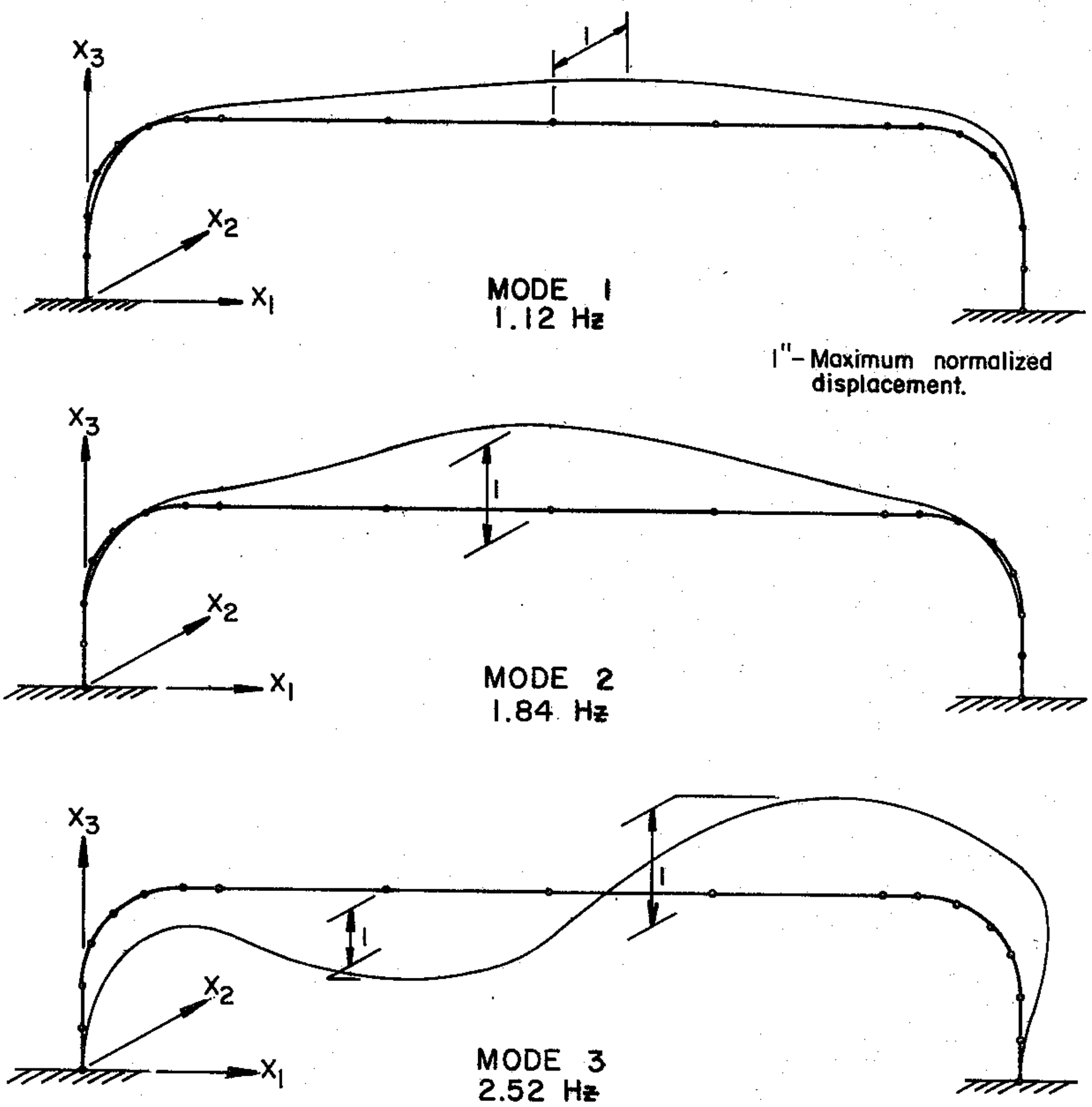
TRAFFIC SIGNAL BRIDGE

Figure 3

TEST B INSTRUMENTATION LAYOUT



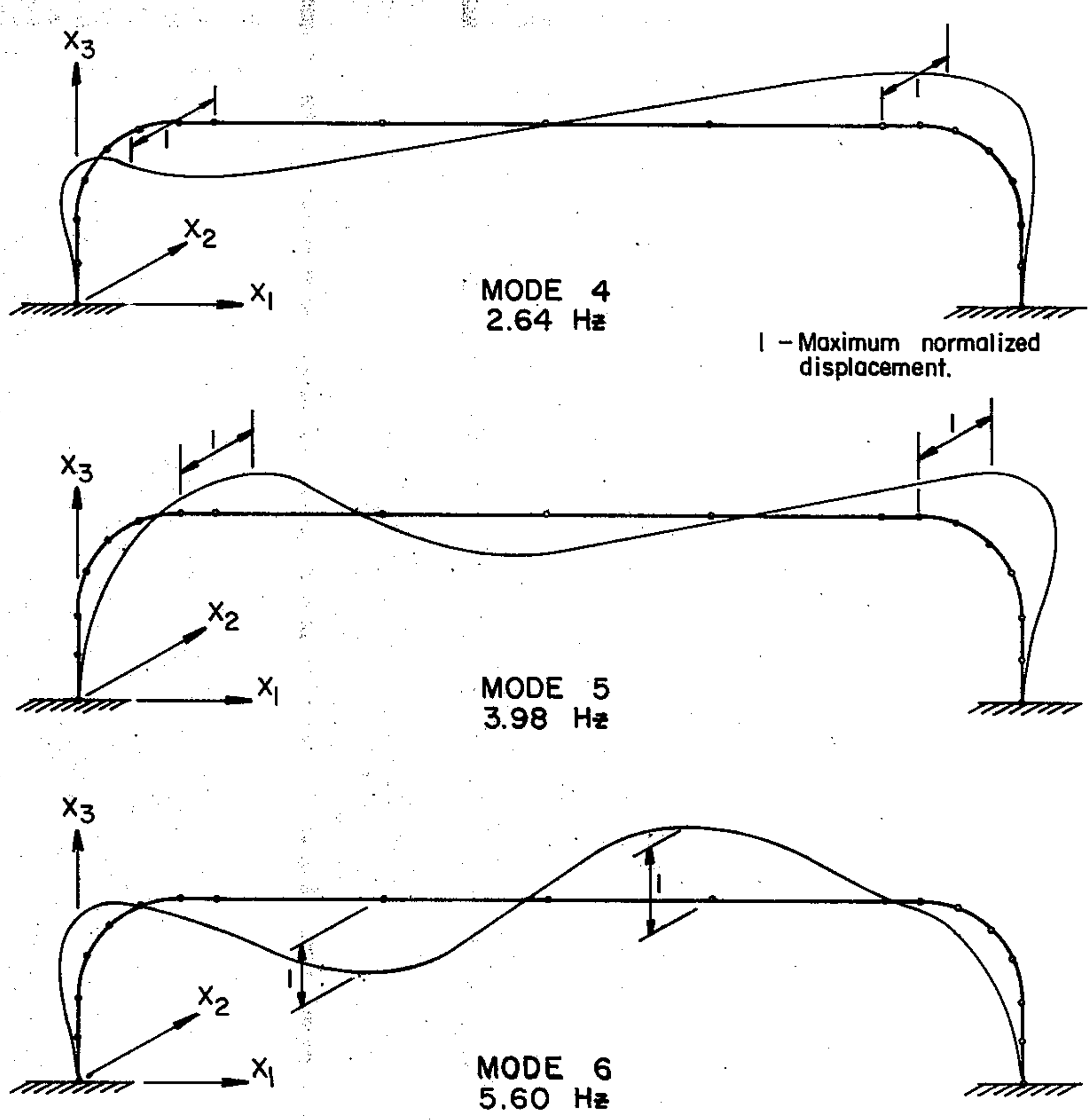
TRAFFIC SIGNAL BRIDGE
Figure 4
ANALYTICAL MODELS FOR EIGENVALUE DETERMINATION



TRAFFIC SIGNAL STRUCTURE

Figure 5

THEORETICAL MODE SHAPES 1,2,3



TRAFFIC SIGNAL STRUCTURE

Figure 6

THEORETICAL MODE SHAPES 4,5,6

Appendix A

Calculation of Structural Damping

The structure was excited at a natural frequency to a steady state condition and the driving force was then removed. The structure's free vibration displacement and acceleration curves were recorded on an oscillograph. The logarithmic decrement was then calculated from the decay curves.

It can be shown that, for an underdamped structure,

$$\ln\left(\frac{\nu_n}{\nu_{n+N}}\right) \doteq \xi N 2\pi$$

where ξ = critical damping ratio

ν_n = displacement amplitude at cycle n

ν_{n+N} = displacement amplitude at cycle n + N

N = number of cycles

Figure A1 presents a typical underdamped free vibration curve.

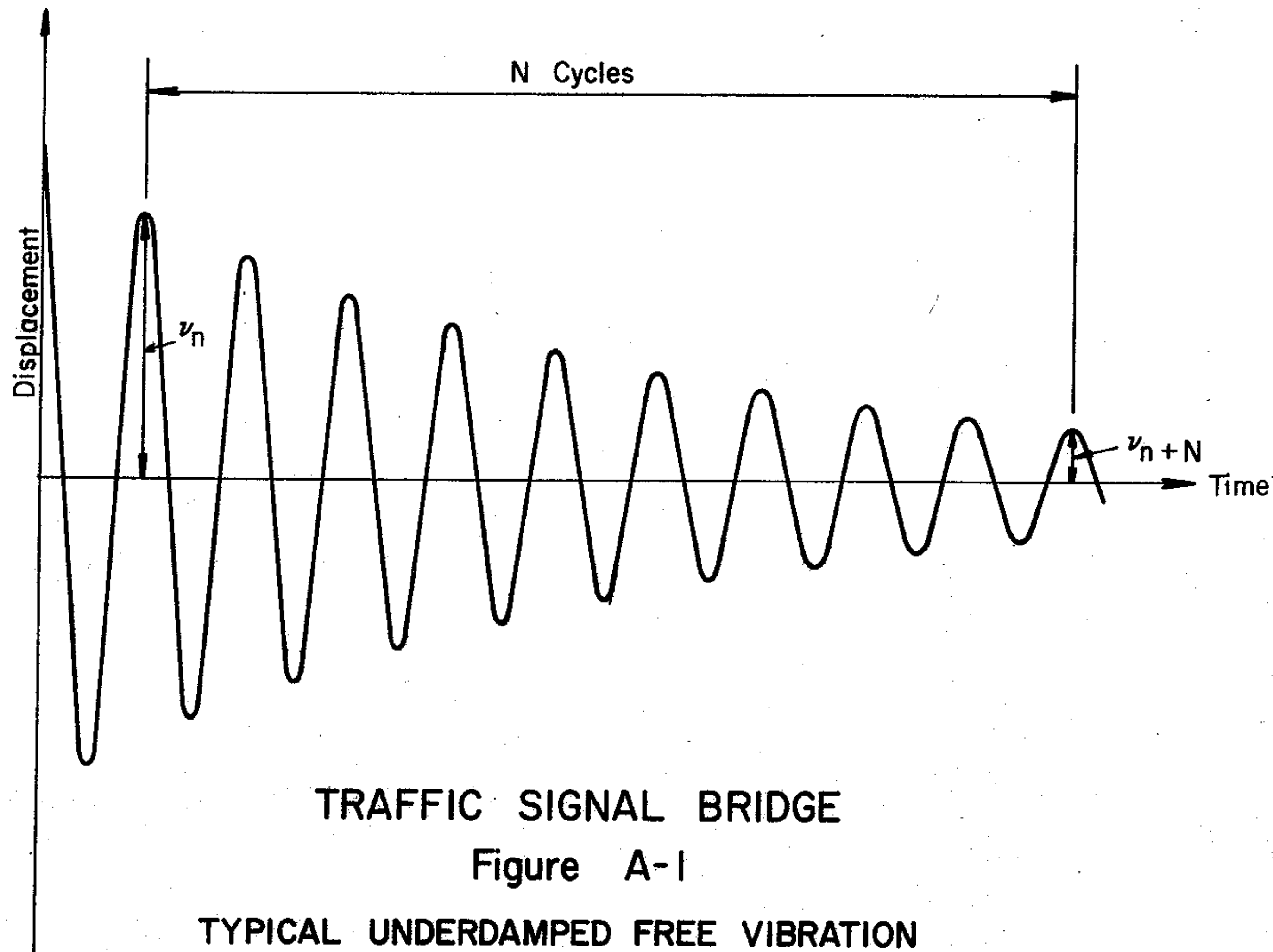
The field produced oscillographs are not included in the report; however, pertinent information from the oscillographs is summarized in Table A1 along with the calculated critical damping ratios.

Mode	Cycle Range	N	ν_n	ν_{n+N}	Damping Ratio X10^{-3}
Mode 3 2.20 Hz	0–50	50	70	30	2.70
	50–100	50	30	15	2.21
	0–100	100	70	15	2.45
Mode 6 4.79 Hz	0–50	50	71.5	36.5	2.14
	50–100	50	36.5	21	1.75
	0–100	100	71.5	21	1.95

TRAFFIC SIGNAL BRIDGE

Table A1

CALCULATED DAMPING RATIOS



TRAFFIC SIGNAL BRIDGE

Figure A-1

TYPICAL UNDERDAMPED FREE VIBRATION
DECAY CURVE