Parametric Study on Human Response to Vibrations of Box Girder Bridges

By

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TO MY PARENTS

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Thanks are due to my parents and my family for their understanding, patience and encouragement.

Special recognition to my teacher Rhoda Diebel for spending her time correcting the English structure and to the computer center personnel.

Acknowledgement is also extended to all my friends who gave me help and advice specially Mr. H. Hachem.

ABSTRACT

Under a moving load a bridge produces deflections and stresses which differ significantly from the deflections and stresses caused by the same load when applied statically. The resulting dynamic deflections can cause discomfort to the pedestrians using the bridge since the human response is related to the acceleration or to the rate of change of acceleration. In the past, several analytical and experimental model studies have been undertaken to study the phenomenon of highway bridge vibrations.

While human discomfort due to perceptible levels of vibration may not be as important in bridge design as human safety consideration, it is a critical factor in bridge serviceability limit and should gain attention accordingly. The literature indicates that researchers tend to place more emphasis on bridge response and behavior for safety consideration and hardly any work on the subject of human discomfort at serviceability.

In general, the human comfort is related to the dominant mode type since the mode type is itself related to the acceleration. As a result, the human body reacts more to vibration of bridge with a torsional dominant mode than those with a flexural dominant mode.

Then, one of the major problems in human comfort study is to avoid this torsional dominant mode.

In this thesis, extensive analytical analyses were conducted to find some parameters affecting the change in the dominant mode type (change from

torsional to flexural and vice versa) and hence reducing the unpleasantness of the bridges' users.
The influence of diaphragms and bracing systems on bridge response is also studied in this thesis.

It is shown that the provision of diaphragms within the boxes at each support or the provision of bracing systems between boxes can reduce considerably the peak accelerations when the dominant mode is torsional. As a result, the human unpleasantness is also reduced.

The linear elastic finite element method was adopted as a method of analysis.

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NOTATIONS

x, y, z	Rectangular co-ordinate system.
u, v, w	Displacements of the nodes in x, y, and z direction respectively.
a	Amplitude.
A	Area.
$[A]$	Matrix contains prescribed functions of (x,y,z).
$[B]$	Strain-Displacement matrix.
$[C]$	Damping matrix of the structure (For dynamic analysis).
C^e	Damping property parameter of element e.
$\{d\}$	Displacement function.
$[D]$	Constitutive matrix.
D_x	Longitudinal flexural rigidity per unit width.
D_{xy}	Longitudinal torsional rigidity per unit width.
E	Young's modulus.
E_c	Young's modulus for the concrete.
$\{F\}$	Vector of forces acting into the direction of the structure global displacements.
$\{F\}_b$	Vector of the element body forces.
$\{F\}_c$	Vector of concentrated loads.
$\{F\}_s$	Vector of the element surface forces.
$\{F\}_\sigma$	Vector of nodal forces due to initial stresses.
$\{F\}_\epsilon$	Vector of nodal forces due to initial strains.

f^b	Vector of body forces.
f^s	Vector of surface forces.
(F)	Flexural mode.
f	Frequency in cycles/sec.
G_c	Shear modulus of the concrete.
I_f	Impact fraction.
I_g	Combined second moment of area of a spine and associated deck slab concrete.
K	Measure used to represent the degree of strain on a person.
$[K]$	Structure stiffness matrix.
L	Span length in meters.
$[M]$	Mass matrix of the structure.
(N)	Total number of bracings.
$[N]$	Shape function matrix.
n_s	Ratio of shear moduli for steel and concrete.
P_v	Distance between the centerline of the boxes.
R	Aspect ratio (span/width ratio).
t	Thickness of the diaphragm.
t	Time variable for forced vibration analysis.
t_0	Time constant.
t'	Different thicknesses used in the contour integral.
(T)	Torsional mode.

$\{U\}$	Vector of the system global displacements.
$\{\dot{U}\}$	Vector of the nodal point velocities.
$\{\ddot{U}\}$	Vector of the nodal point accelerations.
$X(x)$	Polynomial used in finite strip method.
$Y(y)$	Characteristic beam function for finite strip method.
$\oint ds/t'$	Contour integral along the meridian line of the reciprocal of the wall thicknesses.
$\{\alpha\}$	Displacement parameters for the element.
ν_c	Poisson's ratio for the concrete.
ρ^e	Mass density of element e.
ϕ	Vector of order n.
$\{\Phi\}$	Transformation matrix.
ω	Frequency of vibration (rad/sec).
$[\Omega^2]$	Diagonal matrix storing the eigenvalues ω_i^2 on its diagonal.

Chapter 1

INTRODUCTION

1.1 General

Various types of bridge systems have been introduced into North America, Europe as well as in many other parts of the world due to the tremendous development of highway networks. Among these types, the box girder is the most widely used. The single or multicell reinforced, prestressed concrete and composite box girder bridges were first developed in Europe, about forty years ago.

These box beam or spine beam bridges consist of interconnected plate elements of either reinforced or prestressed concrete, or structural steel or a combination of both and provide reasonable flexural and torsional strengths to resist the applied loads.

Actually, they are economically very competitive with other forms of bridge construction because of their adaptability to the complex geometry involved in the highway systems. They can be constructed to follow any desired alignment in plan. The completed structures are also aesthetically pleasing and require minimum maintenance. In addition, these box girder structures are particularly well adapted to carrying pipelines, conduits, cables and other utilities. The large cells provide adequate space for numerous utilities that are kept completely out of view. Some box girder bridges have used the cells for carrying large amounts of drainage, either in large pipes or, in some cases, by using the girder cells as box culverts.

All the above qualities have led to the increased use of box beams in North America over the past two decades. For spans between 60 ft (18 m) and 100 ft (31 m), cast-in-place multicell reinforced concrete box girder bridges which are straight, skew, curved or of arbitrary geometry in plane are widely used for overcrossings, undercrossings and interchange structures. For spans between 90 ft (27 m) and 150 ft (46 m) the composite concrete deck-steel box girders have been used. For longer spans, up to 200 ft (61 m), post-tensioned prestressed cast-in-place multicell box girder bridges are often used whereas, for spans between 200 (61 m) and 800 ft (244 m), commonly used in viaducts or water crossings, segmentally erected prestressed concrete box girder bridges of one, two or more cells may be used.

This thesis explores the linear dynamic analysis of simply supported composite concrete deck-steel box girder bridges using the ADINA (Automatic Dynamic Increment Nonlinear Analysis) program.

A typical composite box girder bridge consists of a cast-in-place concrete slab connected to three-sided-steel boxes through appropriate shear connectors to form a cellular structure (see fig 1.1). Usually diaphragms are present in these bridges specially at the ends and at the points of intermediate support for continuous bridge. Besides, in most cases intermediate diaphragms are also provided between boxes.

The open three-sided-steel box can undergo considerable distortion or twist during the construction process because of construction loads consisting of the wet concrete, formwork, concrete finishing machine and the occasional wind loads. Before and during the pouring of the concrete deck, excessive changes in the cross-section geometry have been observed [27]. To increase the torsional and distortional stiffness of the open section during construction, bracing systems should be installed. These bracing systems, such as ties, distortional bracing, torsion boxes and top chord bracing are usually installed within the girder or between adjacent girders. However, design or construction engineers are not provided with sufficient guidance regarding the behavior of box girders during construction because the emphasis in design is on the bridge once in service. Then it's up to the general contractor or steel fabricator to provide bracing.

Several studies were conducted to investigate the effect of bracing systems on box girder bridges for either static or dynamic analysis. Branco and Green [6] developed an experimental-analytical research program to analyse statically the influence of bracing systems on open section box girders. Mirza et al [21] studied the influence of bracing systems on dynamic responses of composite bridges. However, in the latter study, the investigations of bracing effects are restricted to only flexural dominant modes.
To the best of my knowledge, no studies were conducted specifically for torsional modes and their effects on human responses to bridge vibrations.

In general, complaints from individuals concerning excessive vibrations usually express the opinion that the structure is not safe even though in the structural safety point of view, the stress levels in bridge components resulting from the applied loading are well within the allowable limits. On order to resolve this problem, excessive dynamic deflections or accelerations must be avoided so that reasonable degree of comfort to pedestrians using the bridge can be achieved.
Usually dynamic deflections are more pronounced for the bridges with torsional dominant mode especially at the sidewalk locations. Therefor, due to large dynamic amplifications, people are more sensitive to torsional mode of vibrations than those of flexural dominant modes.

The governing mode is also directly related to the acceleration resulted from the response of the bridge. For bridges with torsional dominant mode the acceleration values are higher than those with flexural dominant mode.

Since people always feel unpleasantness with higher accelerations, the torsional mode must be avoided at all cost.

Most of the previous studies on bridge dynamic response seems to be all focussed on the flexural dominant mode. No attempts were made to study governing parameters affecting the change in the dominant mode types (change from torsional to flexural) and to see the influence of diaphragms or bracing systems on the bridge response when the dominant mode is torsional.

1.2 Objective and Scope

The objective of this thesis is to determine the different parameters affecting the change in the dominant mode shape (first mode) from a torsional one to a flexural one or vice versa. The objective also includes the influence of diaphragms and bracing systems on the dynamic response of a simply supported composite concrete deck-steel box girder bridge when the dominant mode is torsional.

The scope of this work is restricted to straight, simply supported twin box girder bridges. The analytical procedure utilizes the linear finite element method using the ADINA program. The choice of the method was motivated by economic considerations since everything is available at the computer center of the university.

1.3 Outline of the Thesis

The existing literature on bridge vibrations is briefly reviewed in chapter two. In chapter three, the human response to vibration is summarized indicating the bridge vibration tolerance level used until now. Chapter four is devoted to listing the assumptions and to formulating general equations representing the analytical study. And since the ADINA program is mostly used in this thesis, a description of this program and its validity is also given in this chapter.

Chapter five outlines the analysis of box girder bridges. A free vibration study is conducted to determine the different parameters affecting the change in the type of dominant mode (change from flexural to torsional or vice versa). This is followed by a forced vibration study to determine the change in the levels of peak accelerations for box girder bridges due to the presence of diaphragms or bracings. Results are discussed in this chapter.

In chapter six conclusions are drawn. And they are followed by some recommendations concerning the response of box girder bridges.

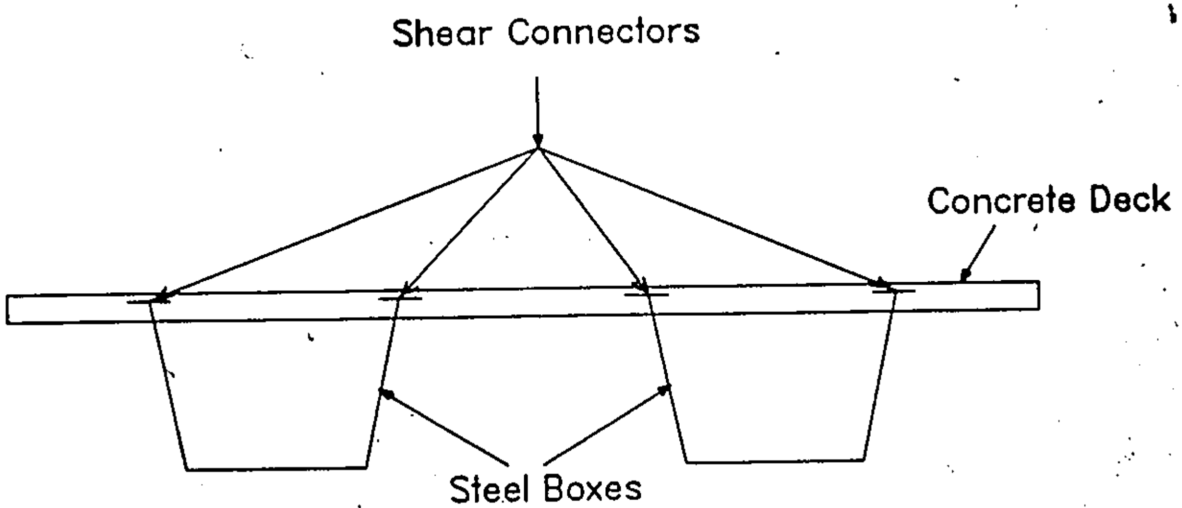


Figure 1.1: Composite Concrete-Steel box girder section (Typical Section)

Chapter 2

LITERATURE REVIEW

For a long time, before the tremendous development in computers, the bridge was idealized as a beam. Many studies on dynamic behavior of single span, constant thickness bridges were based on this idealization.

Among the authors, Inglis [17], Hillerborg [14] and Biggs used this assumption which is valid only for relatively long span bridges.

Tan et al. [29] have also idealized a curved box girder bridge as slender curved beam subjected to either a moving force or a sprung mass to simulate a vehicle. Louw [20], idealizing his bridge as a prismatic beam, came out with an approximate method by which the frequencies and corresponding peak dynamic deflections of simple highway bridges under the action of a two axle vehicle could be predicted. Using the Levy approach, Huffington and Hoppman [16] determined the exact frequencies and modal eigenfunctions for single span rectangular orthotropic plates with 'bridge type' boundary conditions by direct solution of the governing differential equation of motion.

In 1958, Yamada and Veletsos [33] studied the vibration of single span I beam and slab bridge decks treating the slab as a continuum over a series of flexible beams using the Rayleigh-Ritz method. Using the energy method as well, they solved the free vibration problem idealizing the structure as an equivalent orthotropic plate. Immediately after them, Huang and Walker [15] studied the free vibration of curved box-girder bridges mainly using Vlasov's thin-walled beam theory. H. Yonezawa [34] applied the solutions for orthotropic fan-shaped plates, which are simply supported at two opposite straight edges and rigidly attached to elastic beams at the other curved edges, to the numerical analysis of curved girder bridges for free vibration. Using the Galerkin method, he investigated the influence on the frequency of the mass, the width of the bridges and the difference of dimensions in the cross sections of each girder. He compared his results with the elementary curved beam theory. The moving load was a great problem at that time. In 1966, Walker and Veletsos [31] studied the response of simple span highway bridges to moving vehicles. They idealized the bridge as a single beam with distributed flexibility and concentrated point masses equally spaced along the span. The vehicle was represented as a weight resting on a massless spring representing the suspension system for each axle of the vehicle and a frictional device which simulates the effect of interleaf friction in the suspension system.

Veletsos and Huang [30] did the analysis of the dynamic response of highway bridges idealizing the bridge as a beam having a finite number of degree of freedom. The discretization was effected by replacing the distributed mass by a series of concentrated point masses and considering the beam flexibility to be distributed as in the original structure in the manner shown in Fig 2.1 while the vehicle was represented as a three-axle sprung load, with due account taken of the effect of interleaf friction in its suspension system (see Fig 2.2).

Csagoly, Campbell and Agarwal [10] conducted a bridge vibration study based on the beam theory. They obtained good accuracy for the natural frequencies of a continuous beam by means of the STRUDL II dynamic analysis program using a five-element subdivision per span and a lumped mass model. Billing [5], in his report, presented tables of natural frequencies for straight, continuous symmetric multispan uniform beams of two through six spans, for various span ratios. And a simple method for the idealization of a bridge as a uniform beam was given.

Cheung and Cheung [8] used the finite strip method for a frequency analysis of certain single and continuous span bridges and compared their results with other methods. In this method, the bridge deck is assumed to be an assemblage of parallel orthotropic strips each of which may have orthotropic properties which may differ between strips but are constant within each strip.

A simple polynomial X(x), given in terms of the displacement parameters at the two adjacent nodal lines, and a characteristic beam function Y(y), that satisfies the end conditions of the strip, represent the displacement function chosen for each strip. This function defines the displacement and its derivatives uniquely along the interface which ensures the compatibility between two adjacent strips.

Using also the finite strip method Cheung and Cheung [7] studied the free vibration of curved and straight beam-slab or box-girder bridges. The bridge plates were divided into a number of curved or straight strips extending from one support to the other. By assuming suitable displacement functions for the u, v and w displacements they formulate the stiffness and mass matrices of a strip. Then, the desired frequencies were produced by an eigenvalue solution of the assembled overall dynamic stiffness equation.

The advent of high speed computers during the last two decades has led to a growing popularity of the finite element method. The versality and generality of this form of solution, obtained without solving the governing differential equation directly are some of its main advantages. although the required computer storage may sometimes be a limiting factor.

The method is equivalent to the principle of stationary potential energy and is based on assumed displacement expansion. The structure is represented as an assemblage of a number of finite elements interconnected at a discrete number of nodal points located on their boundaries. The displacements within each element are expressed in terms of nodal degree of freedom considered as the unknown quantities. Then by appropriate replacements of the displacement expansions in the stress-strain relationship and appropriate integrations, the element stiffness, mass and stress matrices can be calculated.

Using the finite element method, R. Raizadeh and S. Shore [25] conducted a dynamic analysis of curved box-girder bridges. Annular plates (Fig. 2.3) and cylindrical shell elements (Fig. 2.4) were used to discretize the slabs, bottom flanges and webs. Rectangular plate elements and pin-jointed bar elements were used for diaphragms discretization (see Fig 2.5) . A vehicle, as the applied time varying forcing function, was simulated by two sets of concentrated forces, having components in the radial and transverse directions and moving with constant angular velocities on circumferential paths of the bridge. M.S. Cheung, Chan and Beauchamp [9] carried out a dynamic measurement program on a number of steel-concrete box-girder bridges with different span lengths and number of spans. The program involved the multi-point measurement of strain and acceleration at selected locations in the bridges due to normal traffic and special test vehicles. The data acquired were used to determine bridge dynamic characteristics such as natural frequencies, modal shapes, impact factors and damping ratios.

S.Mirza et al., in March 1985 [21], presented an analytical-experimental study of the dynamic behavior of a composite concrete deck-steel box girder bridge. Their analytical study was based on the finite element method using the SAP IV (Structural Analysis Program) program to analyse the bridge.

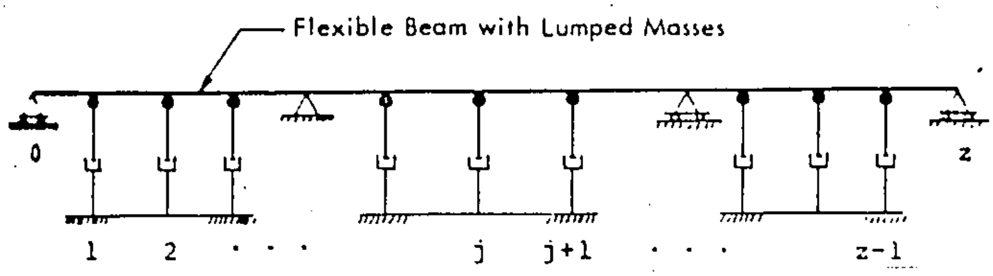


Figure 2.1: Idealization of bridge

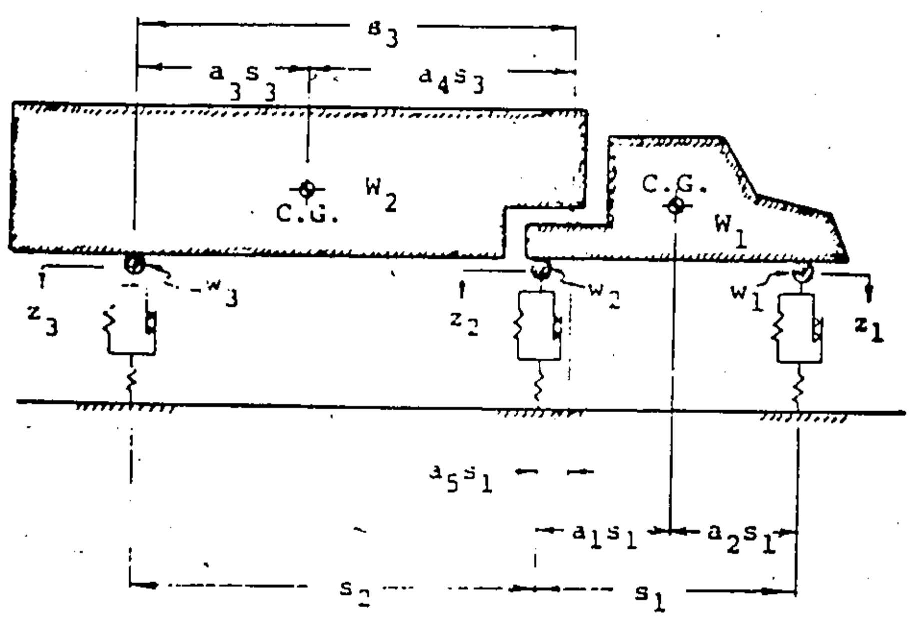


Figure 2.2: Idealization of vehicle

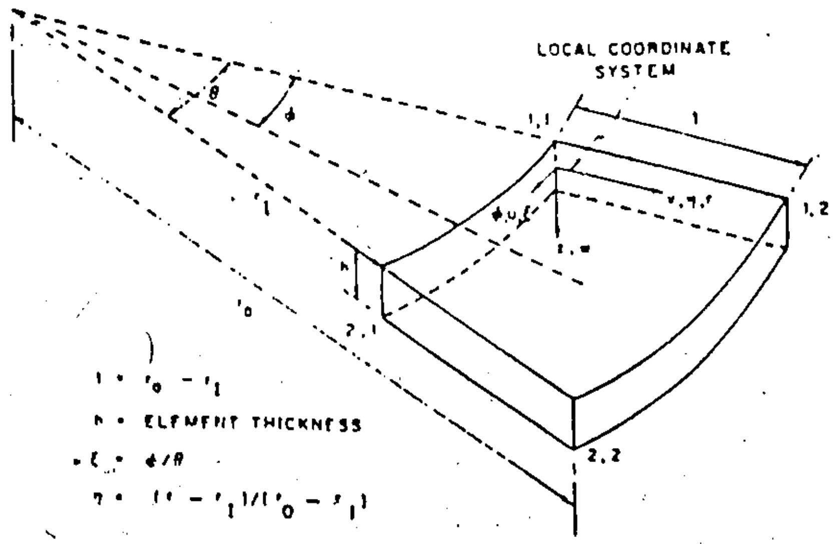


Figure 2.3: Annular Plate Element

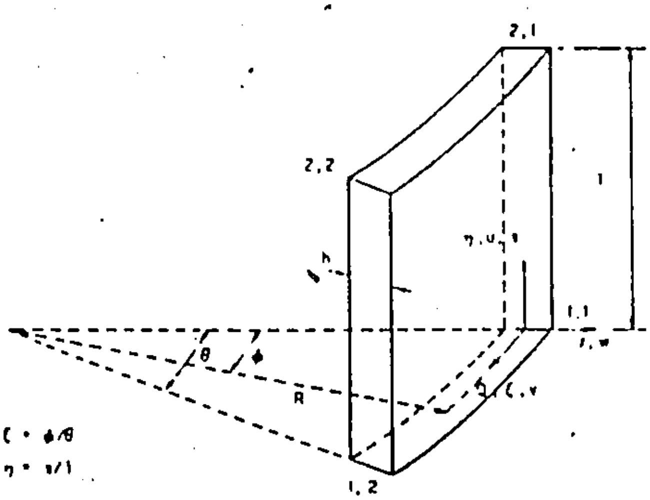


Figure 2.4: Cylindrical Shell Element

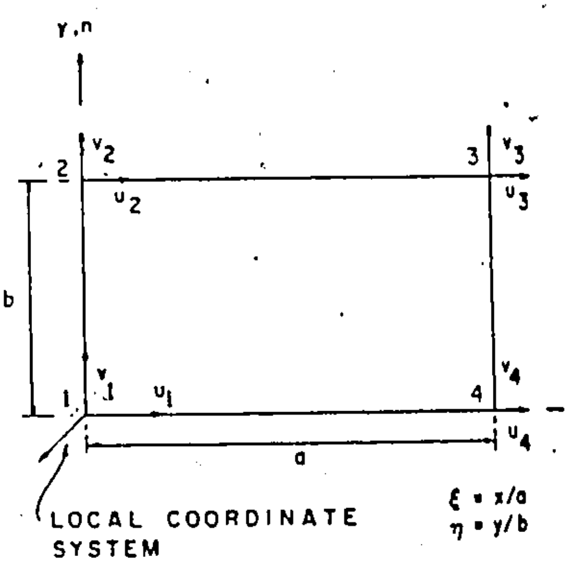


Figure 2.5: Rectangular Plate Element

Chapter 3

HUMAN RESPONSE TO BRIDGE VIBRATION

Many studies were conducted in the last decade to examine the human reaction to bridge vibration and to determine acceptable limits up to which designers can allow bridge movement. These limits are directly related to the reaction of pedestrians. In general, individuals vary in their reactions to vibration. The two main factors, amplitude and period, combined with the varying sensibilities of individuals make it impossible to establish exact limits for perceptibility.

The human body is so sensitive to vibrations that, in general, limits of tolerance are reached at levels which are less severe than would normally give rise to distress in bridge structures. These vibrations are often interpreted by the public to be an indication of structural weakness. Complaints from individuals concerning observed deflections and vibrations usually express the opinion that the structure is not safe.

The bridge user can be affected directly by the forces due to wind and this could be a serious problem for him and for the vehicles especially on exposed bridges e.g suspension bridges. But the wind can also act indirectly by exciting the structure itself.

The second kind of dynamic action is the vehicular force. It is the major source of excitation on highway bridges. This force can be accentuated by irregularities in road surfaces and discontinuities at bridge abutments and expansion joints.

The third force being considered as a dynamic force is the pedestrian force. When walking on a bridge, a person can cause a bridge to vibrate, but the movements of a highway bridge due to pedestrians is generally imperceptible. Marching in cadence, however, can cause concern, and marching troops have historically broken step when crossing bridges, especially footbridges.

3.1 Reaction of the Bridge Users

3.1.1 Human Reactions to Vibration

Until now the problem of human reactions to vibration is not completely understood. Even though it is known that the acceleration and jerk have an important effect on sensitivity, we cannot simply express the human reaction in terms of the physical properties describing the motion, the frequency of vibration, the amplitude and the velocity which all contribute to the sensation.

The vibrating force can affect the human body in two ways: physiologically and psychologically.

Physiological effect

Dieckmann [11], Guignard and Irving [13] have shown that the human body is a dynamic system which has natural frequency of its own. The system is very sensitive and is thus able to detect very small levels of vibration. This extreme sensitivity tends to exaggerate personal reactions making individual assessments of intensity unreliable.

Some researchers had selected the frequencies at which feelings of discomfort or resonance occur in the body as follows:

- 1/2 - 1 c.p.s Associated with motion sickness.
- - 2 c.p.s Head resonance (for horizontal movement), also associated with motion sickness.
- 4 - 6 c.p.s Major resonance of the whole body.
- 7 - 9 c.p.s Abdominal resonance (Liver ?).
- 10 - 12 c.p.s Unspecified trunk resonances.

Discomfort or annoyance result if the body is subjected to prolonged vibrations.

Psychological Effects

These types of effects are more difficult to define than the others and vary markedly between individuals. Vibration can affect people's attitudes, feelings and work performance.

Associated with both the physiological and psychological aspects is the problem of fatigue and interference with task performance after long periods of subjection to vibrations. Much work has been done on these aspects especially in the fields of land and air transportation. this same problem should not arise for the bridge user when crossing the bridge.

3.1.2 Bridge Users

A distinction must be made between two types of bridge users: the pedestrian and the passenger in the vehicle.

Pedestrians

Amongst those who use the bridge, the pedestrian will have the greatest reaction to vibration and will directly influence any design limitations. Tests showed that for a standing person there is a marked reduction in tolerance level compared to that of a walking person on a bridge. This is due to the flexing of knees in the process of walking which creates a hinge effect and so reduces the amount of movement transmitted to the body.

When walking frequency and bridge response frequency are coincident, interference can occur. The phasing between the footsteps and the rise and fall of the deck provides an irritating or excessive feeling, which tends to be disturbing rather than painful or uncomfortable. But, because of the difference in walking rates (paces/second), this effect varies with each individual. For example, people who walk with long striding steps are often unaware of any structural movement at all while others show adverse reactions to the same level of vibration.

Passengers in Vehicles

It is not likely that occupants of vehicles will be troubled by vibrations because of :

- -The short length of time that the vehicle is on the bridge.
- -Vibrations due to uneven road surfaces occurring before riding over the bridge structure.

Hence bridges will be crossed almost unnoticed. However, if the vehicles are parked on bridges or flyovers the movement of vehicles in other lanes of the deck can cause vibration of the structure which is magnified due to resonance with the vehicle's suspension system. Such movements lead to a feeling of motion sickness and give the impression that the bridge structure is moving excessively.

3.2 Human Vibration Studies

A lot of work was done previously on human reactions to vibrations to establish levels of human tolerance to any vibration.

For the first time , in 1931, Reiher and Meister [26] produced six tolerance ranges based on the reactions of 25 adults who were subjected to a sinusoidal movements in the vertical or horizontal directions for a period of 10 minutes while standing, sitting or lying on a platform. As shown in Fig 3.1 the tolerance ranges are classified as imperceptible, just perceptible, annoying, unpleasant or disturbing and painful.

In 1948 Goldman [12], after reviewing the problem, produced a set of averaged curves corresponding to the three tolerance levels of: perception, unpleasantness and intolerance. His study was summarized in graphs given in Fig 3.2 and 3.3 which help to provide an order of magnitude of intensity against frequency in the absence of any specific information relating to a particular problem.

Ten years after Goldman, Dieckmann [11] produced another set of curves defining tolerance levels. Figures 3.4 and 3.5 describe four tolerance levels:

- - K=0 Lower limit of perception.
- - K=1 Allowed in industry for any period of time.
- - K=10 Allowed only for a short time.
- - K=100 Upper limit of stain for the average man.

Where K is a measure used to represent the degree of strain on a person.

Further work was done by Janeway [18] on tolerance level, ending with the use of curves defined by :

- $a\,f^3 = 2$ (1 *to* 6 *c.p.s*)
- $a\,f^2 = 1/2$ (6 *to* 20 *c.p.s*)
- $a\,f = 1/60$ (*above* 20 *c.p.s*)

Where a: amplitude in inches
f: frequency in cycles/sec.

This curve, given in Fig 3.6 was derived for riding comfortably in vehicles.

In Canada, Wright and Green [32] took measurements on 52 bridges and interpreted their findings in terms of levels on the Reiher and Meister scale and on the "isosensor" scale which they had derived from Goldman's work (see Fig 3.7). Corresponding to the peak levels of the vibrations, measurements showed that the "unpleasant" level was frequently encountered and that over 25% of the bridges reached the 'intolerable' level. It should be noted from their results that avoidance of low natural frequencies of bridges up to 3 c.p.s is not in itself sufficient to minimize bridge vibrations.

In the United States, Oehler [22] carried out comparative tests on 15 bridges of three types: simple span, continuous span and cantilever. Results showed that cantilever bridges were more prone to vibrations than the other types. He concluded his study by indicating that there was a tendency for response of bridges to increase with decreasing natural frequency of the structure.

In December 1965, at the Road Research Laboratory [19], tests were carried out to determine tolerance limits for pedestrians walking and standing on bridges. The results are given in Figures 3.8 and 3.9 The duration and magnitude of peak vibrations have been found to be of equal importance in determining acceptable levels for vibrations.

3.3 Design Criteria for Bridge Vibrations

In general, specific design for the avoidance of vibration problems in bridges has been neglected or ignored, control being "exercised" indirectly by design criteria such as stiffness and impact allowance. Generally, the stiffness criteria was stated implicitly in terms of restrictions on "live load deflection" to span ratios and recommended limits for structural depth to span ratios. This live load deflection was, in fact, the static deflection due to a standard load increased by a factor, called impact factor, to allow for "dynamic effects" in the structure. So, the dynamic characteristics of bridges namely, frequency, acceleration, amplitude, damping and true stresses, were avoided almost completely in the dynamic design of highway bridges.

Actually most countries use the impact factor in their codes to allow for dynamic stress. The American AASHTO Specification [28] used to limit the live load deflection/span ratios, where the live load deflection includes the use of a dynamic impact factor and span-depths ratios. They described the complex interaction between a bridge and traversing vehicles by the following formula developed in the late 1920s :

$$I = 15/(L + 38)$$

Where

- - I : Impact fraction, not to exceed 0.30
- - L : Span length in meters.

The allowance for dynamic vibratory or impact effects is applied as a fraction of the live load stress in a component which can be taken as zero. But confusion can occur as to which component should have the static effects of live load stress amplified for design purposes. The Ontario Highway Bridge Design Code [23] describes the dynamic load allowance in terms of an equivalent static load which is a fraction of the OHBD load.

For highway and pedestrian bridges the OHBDC considers the acceleration due to a representative load traversing the superstructure as a serviceability limit state. It does not have any limitations on the depth-to-span ratio, or on the live load deflection as a function of span. These limitations which have been used to control vibrations, prevent fatigue, limit stress in secondary members and account for dynamic loading, are provided for explicitly in the OHBDC. But to simplify the design process, acceleration limits have been converted to equivalent static deflection limits.

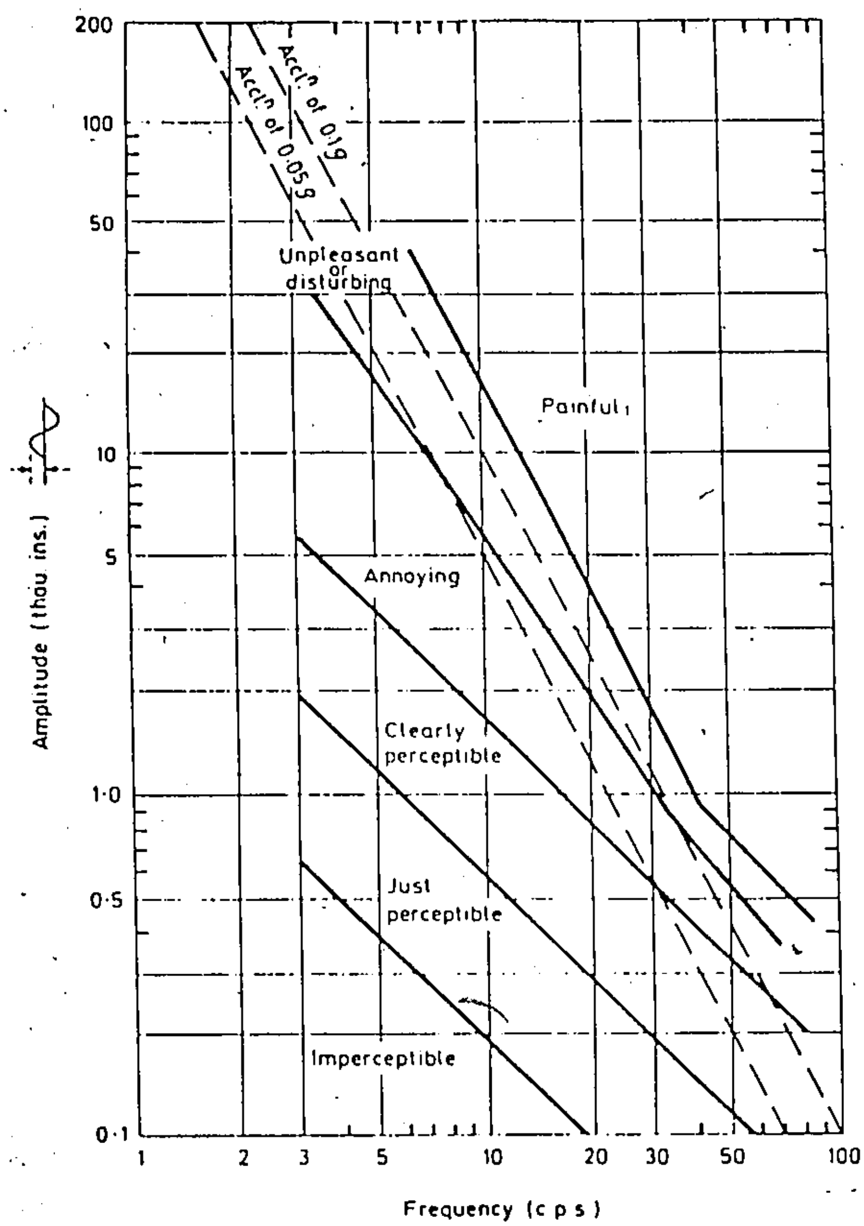


Figure 3.1: Human Sensitivity to Vertical Vibrations (Reiher and Meister)

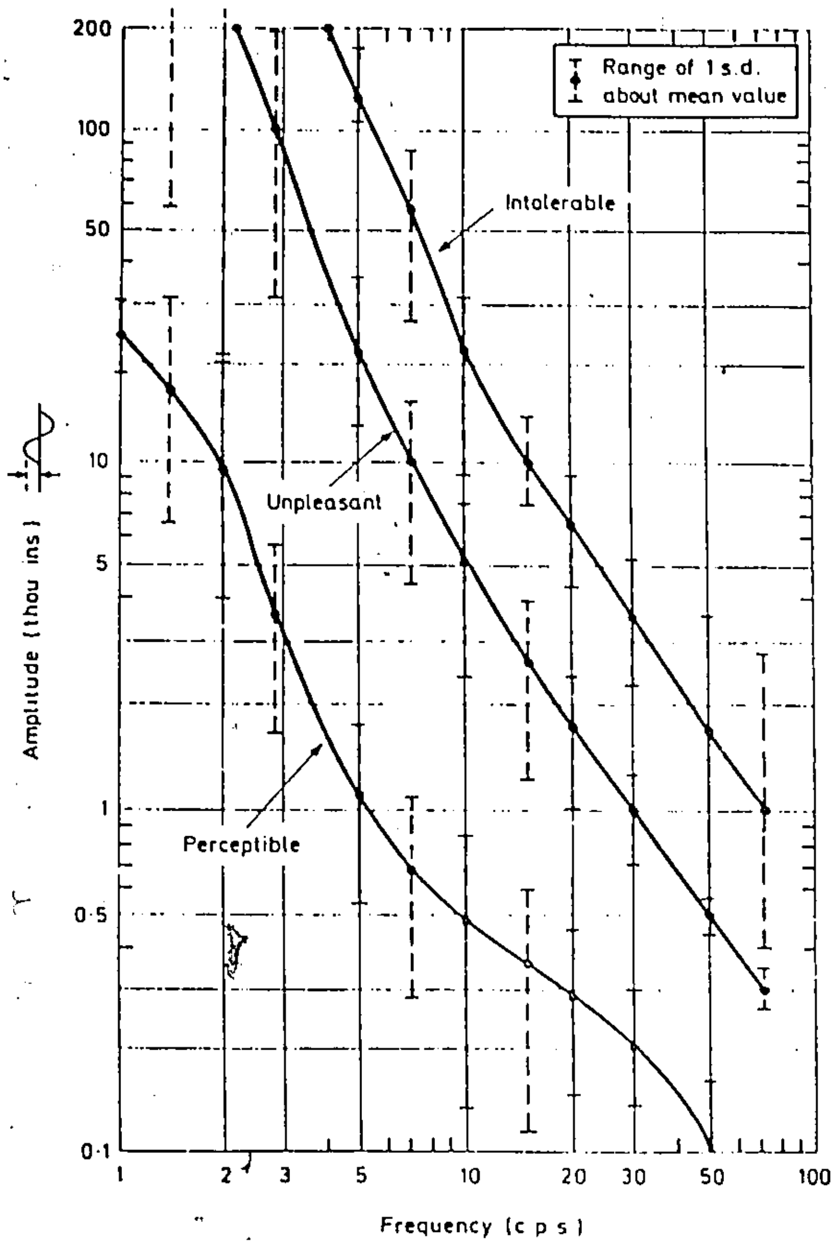


Figure 3.2: Tolerance Levels of Human Reaction to vibration (Goldman)- Amplitude vs Frequency

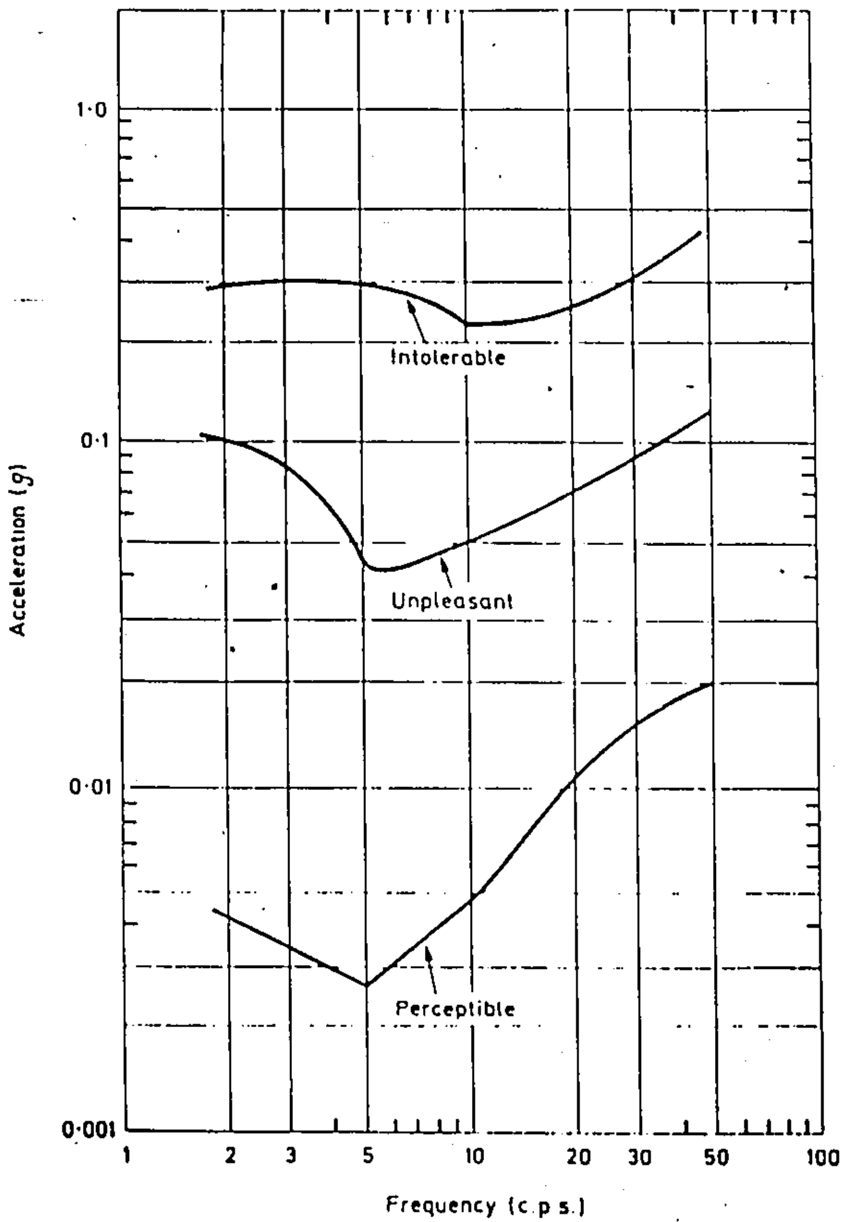


Figure 3.3: Tolerance Levels of Human Reaction to vibration (Goldman)- Acceleration vs Frequency

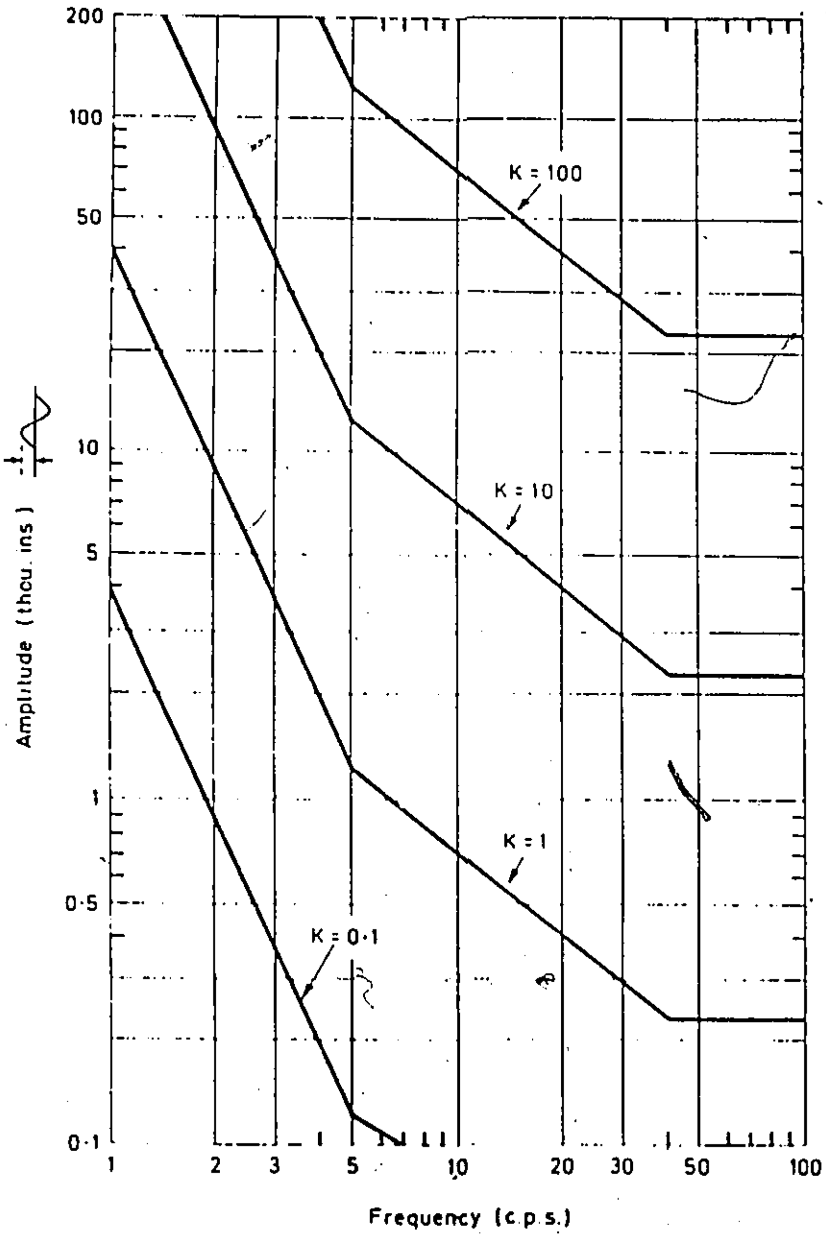


Figure 3.4: Scale of Strain For Vertical Vibrations (Dieckmann)

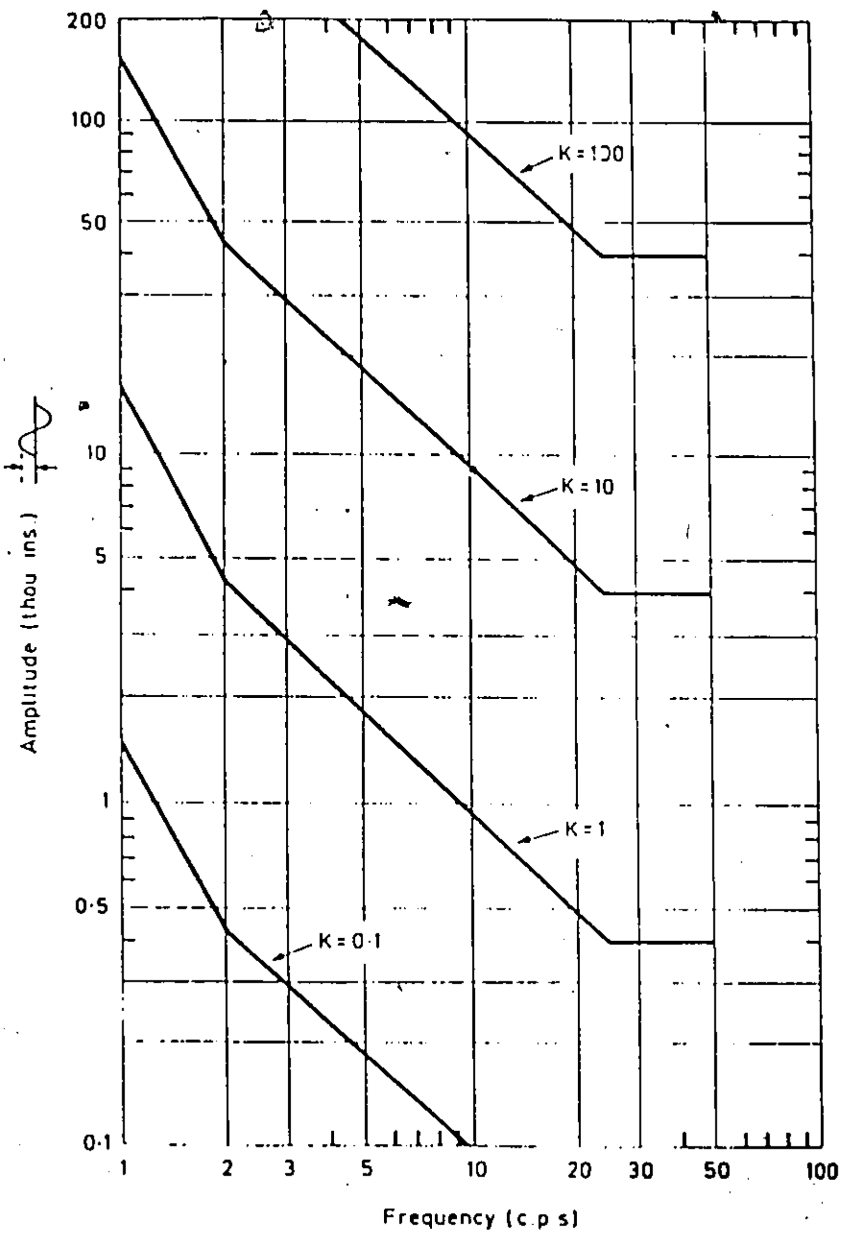


Figure 3.5: Scale of Strain For Horizontal Vibrations (Dieckmann)

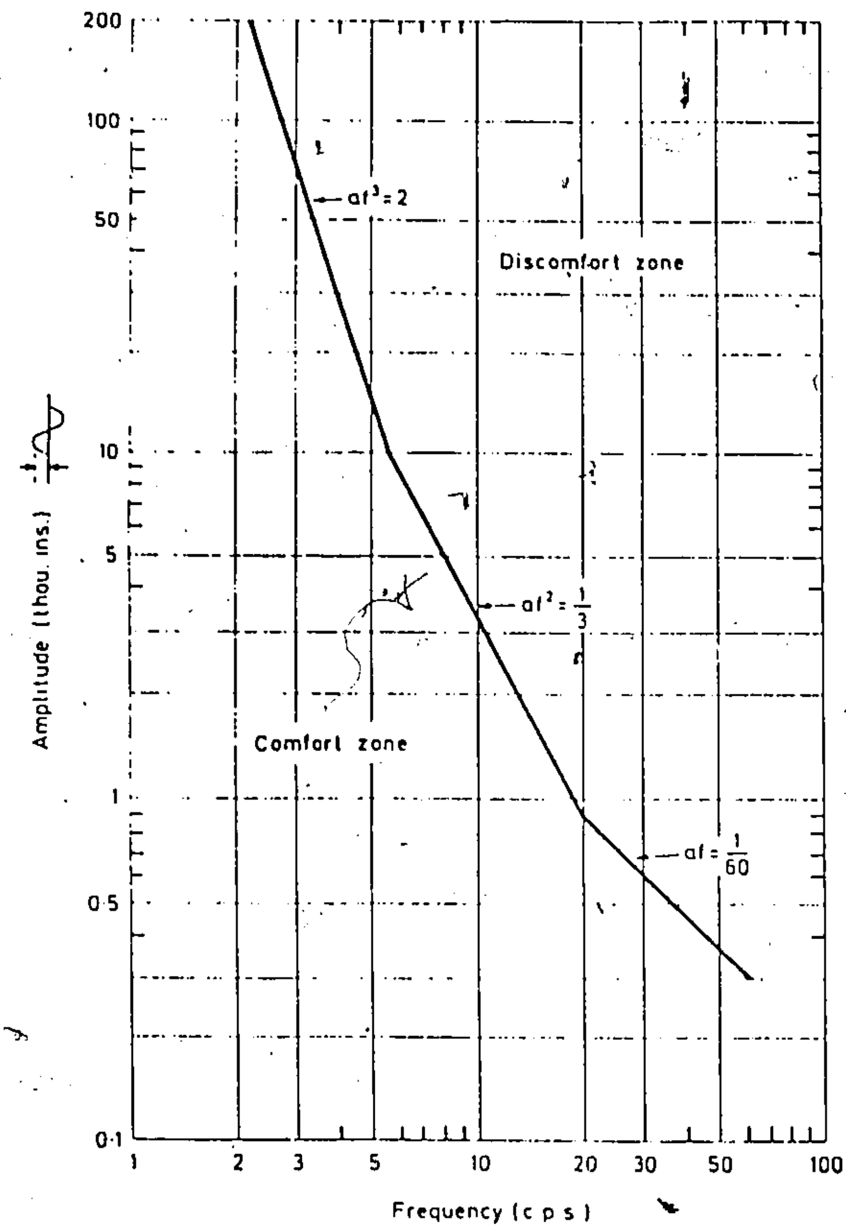


Figure 3.6: Vertical Vibration Limits For Automobile Passenger Comfort (Janeway)

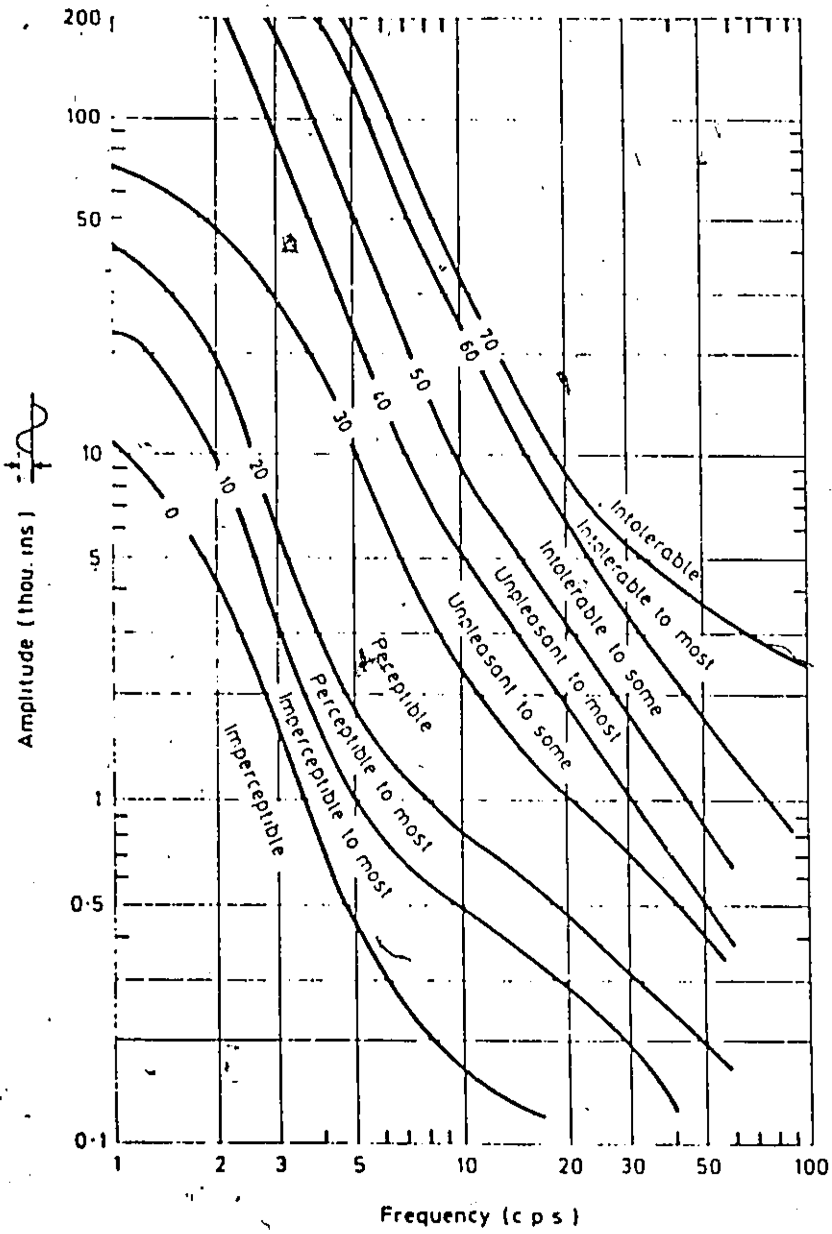


Figure 3.7: Contours of Equal Sensitivity To Vibration-'Isosensors' (Wright and Green)

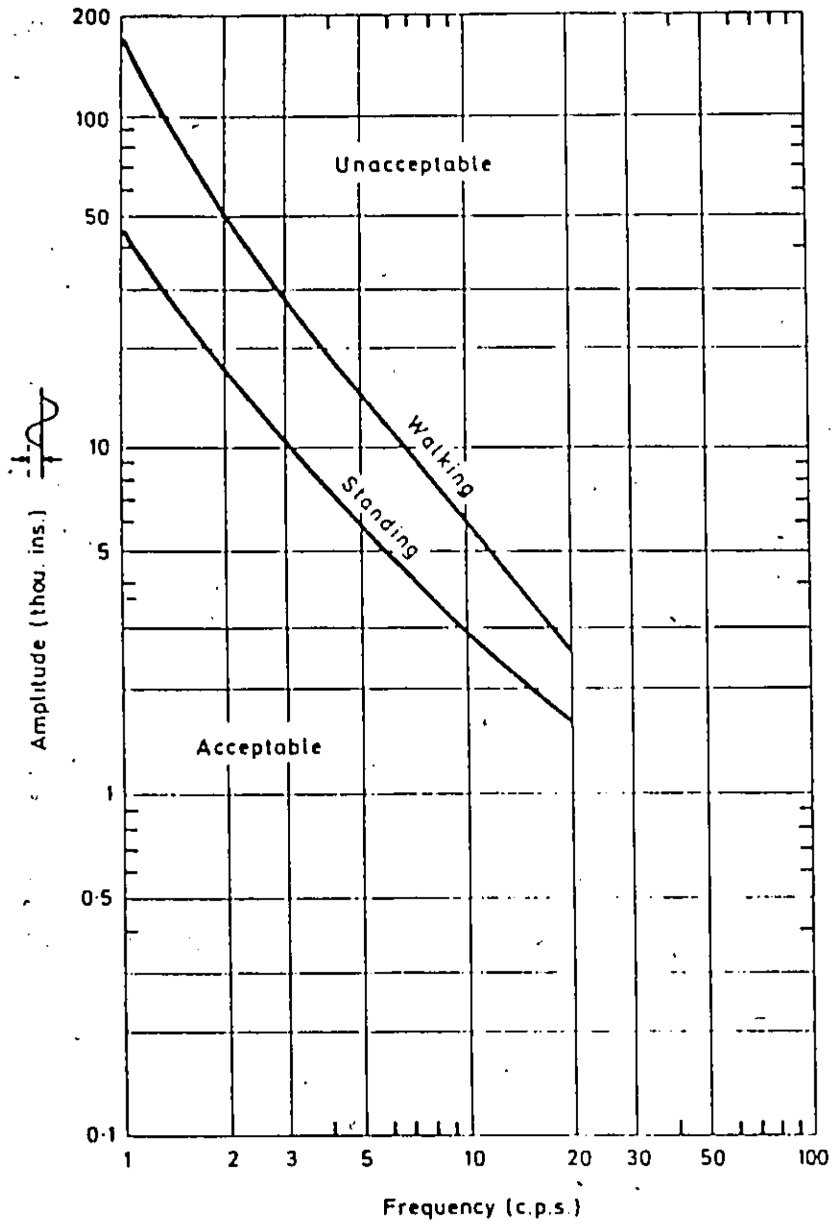


Figure 3.8: Human Tolerance Levels For Bridge Vibrations (Results of Tests. December 1965)

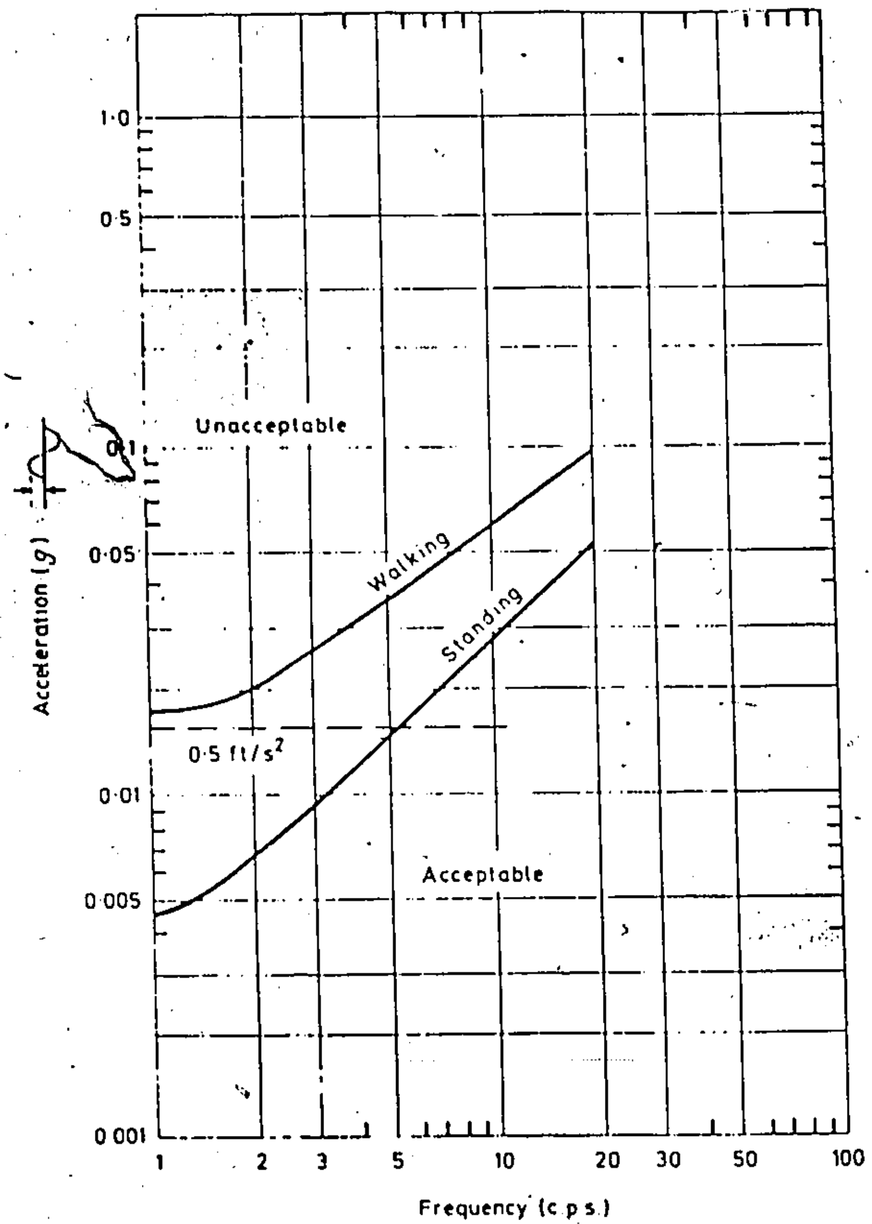


Figure 3.9: Human Tolerance Levels For Bridge Vibrations (Expressed in Terms of g)

Chapter 4

ANALYTICAL PROCEDURE

4.1 General

As stated earlier in this thesis, the finite element method will be used to develop an analytical model to determine the natural frequencies and the response of the composite concrete-steel box girder bridge. The theory and application of the method will be discussed in the subsequent sections.

The composite box girder bridge has two main components, concrete deck and steel boxes. The modelling of each component, based on the behavior of the complete structure, will be described later in this chapter.

There are several special purpose programs which have been developed by various investigators to analyse the complex response of box girder bridges. The development of such a special purpose program requires a considerable expenditure of both financial and human resources.

In the present study, the commercially available program ADINA was used because of its relatively low operational cost and its simplicity of application. The use of ADINA program and its details will be given in subsequent sections.

4.2 Finite Element Method

4.2.1 Introduction

A finite element is a subregion of a discretized continuum. It enables us to model a problem with an infinite number of degrees of freedom to one with a finite number in order to simplify the solution process. It is a computer-oriented method with the primary objectives to approximate the real conditions of the stresses and deflections in a structure.

The classical approach for analyzing a structure requires finding a stress or displacement function that satisfies the equations of equilibrium, the stress-strain relationships, and the compatibility conditions at every point in the continuum, including the boundaries.

The finite element approach yields an approximate analysis based upon an assumed displacement field, a stress field, or a combination of these within each element. Since the assumption of displacement function is the technique most commonly used, the method involves replacing the continuum by an assembly of discrete or finite elements which are interconnected normally at nodal points.

The response of the concrete deck is complicated by cracking and subsequent redistribution of internal forces which are time-dependent because of shrinkage and creep in the concrete. These complications become more pronounced at higher load levels. A complete response analysis of a composite concrete deck-steel box girder bridge can be very complex, time consuming and expensive. Elastic analysis of reinforced concrete systems such as frames, shear walls, slabs, shells etc... is used more frequently to determine the internal forces, moments and deformations for design purposes. These elastic analyses assume the concrete section to be uncracked, homogeneous with a linear elastic response; this is a good approximation since for linear dynamic analysis the effect is not significant.

The response of a box girder bridge can be obtained by modelling the bridge using shell elements. The concrete deck, steel girders and diaphragms are idealized as linear elastic shell elements which consist of a quadrilateral element of arbitrary geometry with five degrees of freedom per node whereas the bracings are idealized as a beam element with six degrees of freedom.

The stiffness matrix of the shell element is selected to simulate closely the behavior of the continuum on the basis of the assumed displacement or stress fields which are continuous within the element. The element stiffness matrices are then assembled to form the stiffness matrix for the entire bridge system. Knowing the generalized nodal forces resulting from the applied loading, the generalized nodal displacements can be determined. These are, then, used to determine the strains, stresses and the internal stress resultants.

The accuracy and convergence of the method to the exact solution depend essentially on the number of elements used and on the type of element.

4.2.2 Finite Element Equilibrium Equations

Static Analysis

The finite element is based on the Ritz procedure. The principle of minimization of the total potential energy is used to develop the relationship between unknown nodal displacement parameters and the applied loading.

This is expressed by the following equation:

$$[K]\ \{U\} = \{F\} \tag{4.1}$$

$$with \qquad \{F\} = \{F\}_c + \{F\}_b + \{F\}_s - \{F\}_\sigma + \{F\}_\epsilon \tag{4.2}$$

Where : [K] : Structure stiffness matrix.

{U} : Vector of the system global displacements.

{F} : Vector of forces acting into the direction of the structure global displacements.

$\{F\}_c$: Vector of concentrated loads.

$\{F\}_b$: Vector of the element body forces.

$\{F\}_s$: Vector of the element surface forces.

$\{F\}_\sigma$: Vector of nodal forces due to initial stresses.

$\{F\}_\epsilon$: Vector of nodal forces due to initial strains.

Any chosen displacement function can be written as:

$$\{d\} = [A]\ \{\alpha\} \tag{4.3}$$

Where [A] contains prescribed functions of (x,y,z) and $\{\alpha\}$ contains the displacement parameters for the element.

Going through some changes, The terms of equation (4.1) can be written as :

$$[K] = \sum_e [K]^e = \sum_e \int_V [B]^T[D][B]dV \tag{4.4}$$

$$\{F\}_b = \sum_e \{F\}_b^e = \sum_e \int_V [N]^T \{f\}^b dV \tag{4.5}$$

$$\{F\}_s = \sum_e \{F\}_s^e = \sum_e \int_S [N]^T \{f\}^S dS \tag{4.6}$$

$$\{F\}_\sigma = \sum_e \{F\}_\sigma^e = \sum_e \int_V [B]^T \{\sigma\}_0 dV \tag{4.7}$$

$$\{F\}_\varepsilon = \sum_e \{F\}_\varepsilon^e = \sum_e \int_V [B]^T \{\varepsilon\}_0 dV \tag{4.8}$$

$\{f\}^b$ and $\{f\}^s$ are the vectors of body and surface forces respectively. Having the equilibrium equation (4.1), we can solve for the unknown nodal displacement parameters using the Gauss elimination process or more precisely the Active Column Solution.

Dynamic Analysis

In equation (4.1) the applied forces may vary with time, in which case the displacements vary also with time and equation (4.1) is a statement of equilibrium for any specific point in time. However, if the loads are applied rapidly, inertia forces need to be considered, i.e., a truly dynamic problem needs to be solved.

Also, in dynamic response of structures it is observed that energy is dissipated during vibrations, which, in vibration analysis, is usually taken into account by introducing velocity-dependent damping forces.

These element inertia and damping forces are included as part of the body forces as follows:

$$\{F\}_b^e = \int_V [N]^T \left[\{F\}^b - \rho^e[N]\{\ddot{U}\} - C^e[N]\{\dot{U}\}\right] dV \tag{4.9}$$

$$\{F\}_b = \sum_e \{F\}_b^e \tag{4.10}$$

where: $\{\ddot{\mathrm{U}}\}$: Vector of the nodal point accelerations.

$\{\dot{\mathrm{U}}\}$: Vector of the nodal point velocities.

ρ^e : Mass density of element e.

C^e : Damping property parameter of element e.

The equilibrium equation is, in this case :

$$[M]\ \{\ddot{U}\}\ + [C]\ \{\dot{U}\}\ + [K]\ \{U\}\ =\ \{F\} \tag{4.11}$$

where the matrix [M] is the mass matrix of the structure.

$$[M]\ =\ \sum_e [M]^e\ =\ \sum_e \int_V \rho^e [N]^T [N] dV \tag{4.12}$$

And [C] is the damping matrix of the structure; i.e., formally:

$$[C]\ =\ \sum_e [C]^e\ =\ \sum_e \int C^e [N]^T [N] dV. \tag{4.13}$$

In practice it is difficult, if not impossible, to determine for general finite element assemblage, the element damping parameters, in particular because the damping properties are frequency dependent.

A special case of the dynamic problem is the undamped free vibration system. In this case the equilibrium equation becomes (with [C]={F}=0):

$$[M]\ \{\ddot{U}\}\ +\ [K]\ \{U\}\ = 0 \qquad (4.14)$$

The solution of equation (4.14) is of the form:

$$U\ =\ \phi\ \sin\omega\ (t - t_0) \qquad (4.15)$$

Where ϕ is a vector of order n, t the time variable, t_0 a time constant, and ω a constant identified to represent the frequency of vibration (rad/sec) of the vector ϕ.

Substituting (4.15) into (4.14) we obtain the generalized eigenproblem, from which ϕ and ω must be determined,

$$[K]\ \{\phi\}\ =\ \omega^2\ [M]\ \{\phi\} \qquad (4.16)$$

The eigenproblem in equation (4.16) yields the n eigensolutions (ω_1^2, ϕ_1), (ω_2^2, ϕ_2), ..., (ω_n^2, ϕ_n), where the eigenvectors are M-orthonormalized, that is :

$$\phi_i^T\ M\ \phi_j\ =\ \begin{cases} 1 & \text{i=j} \\ 0 & i \neq j \end{cases} \qquad (4.17)$$

and $0 \leq \omega_1^2 \leq \omega_2^2 \leq \omega_3^2 ... \leq \omega_n^2$.

Defining :

$$\Phi\ =\ [\phi_1, \phi_2, ..., \phi_n] \qquad (4.18)$$

and

$$\Omega^2 = \begin{bmatrix} \omega_1^2 & & & & \\ & \omega_2^2 & & & \\ & & \omega_3^2 & & \\ & & & \ddots & \\ & & & & \omega_n \end{bmatrix}$$

The n solutions to equation (4.16) are :

$$[K]\,\{\Phi\} = [M]\,\{\Phi\}\,[\Omega^2] \tag{4.19}$$

Since the eigenvectors are M-orthonormal, we have

$$\{\Phi\}^T\,[M]\,\{\Phi\} = [I] \tag{4.20}$$

$$\{\Phi\}^T\,[K]\,\{\Phi\} = [\Omega^2] \tag{4.21}$$

The final displacement vectors are written as:

$$U(t) = \{\Phi\}\,X(t) \tag{4.22}$$

Where the components of X are referred to as "generalized displacements".

For the undamped forced vibration (with [C]=0 and $\{F\} \neq 0$) the equilibrium equation becomes:

$$[M]\,\{\ddot{U}\} + [K]\,\{U\} = \{F\} \tag{4.23}$$

The solution of equation (4.23) can be obtained by using either the direct integration or mode superposition method. More details of these two methods can be found in many references [4,35].

4.2.3 Shell Element

With the finite element method, any structure can be modelled using one, two, or three-dimensional elements or a combination of those elements. Figures (4.1) to (4.3) show the commonly used elements.

The problem of selecting the element type for a particular model is mainly based on structural response. It is also based on many factors such as element accuracy, reliability and simplicity as well as computational cost.

One of the most important elements used in bridge superstructure modelling is the shell element. The flat shell element is a 4 to 32-node element which can be employed to model thick and thin general shell structures. The choice of the number of nodes depends on the application itself.

In this thesis the nine-node shell element (Fig 4.4) was used with five degrees of freedom at each node. This was in accordance with the ADINA recommendations on the selection of elements [1,2].
The shell element implemented in ADINA program is an isoparametric element and can be used in the following cases :

1. Linear analysis
2. Materially-nonlinear-only analysis.
3. Total Lagrangian formulation for very large displacements and rotations but small strains.

As a special case of shell element, there is a simple flat shell element which may be regarded as plate element.

The principal idea of the isoparametric finite element formulation is to achieve the relationship between the element displacements at any point and the element nodal point displacements directly through the use of interpolation functions (also called shape functions).
The elements are called isoparametric only when the shape functions defining geometry and function are the same.

It is not necessary to present the formulation of such elements in this thesis, and the reader may find all the theory in reference [4].
However we should emphasize the advantages of the isoparametric shell element used in ADINA which are :

- -Variable thicknesses at the nodes.
- -The geometry of any shell surface can be represented accurately.
- -All displacement compatibility conditions between elements are satisfied directly and in an effective manner.
- -The element can be implemented as a transition element (see Fig 4.5)

But one of the disadvantages of this element, especially for large input data, is the sixth degree of freedom.

4.3 ADINA System

4.3.1 General

The ADINA program is a powerful tool used to analyze any structure by the finite element method. It can be employed effectively for linear, nonlinear, static and dynamic analysis, and can display the input data and solution results.

The structural systems to be analyzed may be composed of combinations of a number of different structural elements. Presently the program contains a variety of elements such as: truss, beam, plate, shell, pipe etc... (see fig.4.6)

The element stiffness and mass matrices are assembled in condensed form and the formation of the structure matrices is carried out in the same way in a static or dynamic analysis.

For a frequency analysis only, the determinant search method or the subspace iteration method can be used while for a dynamic response analysis the direct time integration (Newmark, Central Difference or Wilson theta methods), mode superposition or the response spectrum analysis can be used.

4.3.2 Validity of ADINA

In order to establish the validity of the results from the ADINA program, a comparison with exact theoretical solutions (or other numerical methods) for particular structural systems was done.

The first structural system was a simply supported beam. The beam was idealized using finite beam elements to determine the first four natural frequencies and the modes of vibration. The values obtained from ADINA program using isoparametric beam element are summarized in table (4.1) which show good agreement with the exact values.

The second structural system was a simply supported box girder bridge. To model this bridge a nine node shell element was used. The three first natural frequencies of the bridge are given in table (4.2). These values compared with the beam theory show that there is agreement only for the first mode which is the dominant mode.

The third structural system was a simply supported 2 cell girder bridge. The bridge was analyzed by M.S. Cheung [7] using the finite strip method. Figure (4.7) shows the details of the bridge studied. The same bridge was analyzed by the finite element method using ADINA program (mesh shown in Figure 4.8). The three lowest natural frequencies given in table (4.3) show a good agreement between finite strip method and finite element method.

4.4 Description of Bridges

4.4.1 Material modelling

As described earlier, the bridge is a composite concrete deck-steel box girder. So the constitutive models used for both materials are isotropic linear elastic material models where it is assumed that the total stress is uniquely determined by the total strain. In ADINA, the constitutive relation (material matrix C) is based on the two main material constants:

1. E : Young's modulus.
2. ν : Poisson's ratio.

4.4.2 Bridge Geometry

Since the objective of this thesis is to determine the different parameters affecting the change in the dominant mode shape (first mode), many bridge geometries were used. However, the study of these parameters was undertaken on a specific bridge prototype. The general layout of this type of bridge and a typical cross section are shown in Figure (4.9).
It is a simply supported bridge with two steel boxes equally spaced at 2.45 meters and a concrete deck of 20 cm of thickness. The length is 45.72 meters and the width is 9.35 meters.

The material properties and mass densities for the concrete deck and steel boxes were taken to be the same for all bridges and are given in table (4.4). Figure (4.10) shows the stress-strain relationship of the materials.
The material used for the bracings and diaphragms is the same material used for the steel boxes.

For the finite element idealizations, in general, two meshes were used for the free vibration study depending on the length of the bridge. The first mesh given in figure (4.11) consists of 110 elements and 445 nodes whereas the second mesh, given in figure (4.12), consists of 132 elements and 525 nodes.
For the forced vibration study the mesh is finer than the previous ones (see Fig 4.13) and consists of 198 elements and 777 nodes.

4.4.3 Bridge Bearing

In general, the study was conducted on one type of boundary conditions. One end of the bridge is pinned where there is no longitudinal movement but only rotation. At the other end, there is a roller which permits rotation and longitudinal displacement.
However, in some parametric studies the boundary condition was changed in that both ends were pinned for purposes of comparison.

4.4.4 Bridge Loading

It was stated earlier that for the free vibration study the external force vanishes. On the other hand for the forced vibration study, the load is different from zero. In this case the applied load consists of the OHBD (Ontario Highway Bridge Design) truck shown in figure (4.14). This idealized truck was developed to represent the many types and configurations of real trucks.

The position of the loads in the longitudinal direction is shown in figure (4.15.a). For the purpose of computer use the truck load was approximated by a simplified equivalent load (see figure 4.15.c).This equivalent load consists of two axle loads and produces the same maximum moment (static analysis) as was produced with the truck loading.

For the transverse direction, the truck position is shown in figure (4.16). With the recommendations of the OHBDC, the truck is assumed to be in a travelling lane. However, for comparison of results, the response of the bridge is also given when the truck is positioned at the middle of the bridge (see Fig 4.16.b)

The load is assumed to have no initial vertical motion at the instant it enters the bridge, and its suspension spring is assumed to be blocked so there is no energy dissipation by interleaf friction and its entire flexibility is provided by the tire spring.
Because of time dependant of loads, the bridges were usually crossed by

traffic at low speeds (60 Km/h or less) [23,24]. The modelling of the moving loads depends on the mesh of the structure and on the truck's speed. Figure (4.17) shows an example of time function used in ADINA to model the loads of a truck travelling at 60 Km/h (see also appendix A).

4.5 Parametric Study

4.5.1 General

In any parametric study, the objective is to determine some parameters that affect the analysis of structures. These parameters can be used as axes of design charts or tables. Their existence is of very great assistance in the development of simplified methods of analysis that are based upon results of often complex analysis.

A great economy of time and effort is achieved, if, for many structures such as highway bridges, much detailed analysis can be replaced by the calculation of the values of a few parameters, followed by the use of the charts or tables.

As stated earlier, the main objective of this study is to determine some of the parameters that govern the change in mode type and their effects on the response when the dominant mode is torsional.

The analysis was carried out on the following parameters:

1. Aspect ratio. (span/width ratio)
2. Diaphragms within boxes.
3. Bracing between boxes.
4. Flexural/Torsional rigidities ratio.
5. Truck position (for forced vibration only)

Some of these parameters are used for either the free vibration or the forced vibration studies.

The use of these parameters can be very helpful to solve any vibration problem that may be encountered during a bridge design in the real life. To demonstrate the effect of each parameter, the variation of natural frequency and peak acceleration (for free and forced vibration respectively) with change in these parameters will be plotted and discussed in the next chapter. In the following subsections a brief definition of each parameter is given.

4.5.2 Aspect Ratio

The aspect ratio used here is defined as the ratio of the deck's length to its width. This ratio varies within the interval [1-4.5]. It must be remembered that only short and medium span bridges were considered.

Where considering the aspect ratio, the cross section of the bridge was kept constant. Two different boundary conditions were used for this parameter study for comparison purpose.

4.5.3 Rigidity Parameter

One of the most important parameters in the bridge analysis is rigidity. Two types of rigidity can influence the natural frequency of a bridge: flexural and torsional rigidity.

From previous researches we observed that the relative magnitude of flexural and torsional rigidities have no importance in the change of mode type, the important is their ratio.

The values of these rigidities are governed by the cross section of the bridge. For composite box girder bridge, these values given in reference [3] are summarized in the following sections.

Flexural Rigidity

The longitudinal flexural rigidity, D_x (corresponding to EI in a longitudinal beam) per unit width for a composite twin box-girder bridge is given by the formula :

$$D_x = \frac{E_c}{P_y} I_g \tag{4.24}$$

Where : I_g: is the combined second moment of area of a spine and associated deck slab, in units of deck slab concrete.

P_y: is the distance between the centerline of the boxes.

E_c: Young's modulus for the concrete.

Torsional Rigidity

The longitudinal torsional rigidity D_{xy} (corresponding to GJ in a longitudinal beam) per unit width for a composite twin box-girder bridge is given by:

$$D_{xy} = \frac{G_c}{P_y} \left(\frac{4A^2}{\oint ds/n_s t'} \right) \tag{4.25}$$

* Where A is the area enclosed by the median line passing through the walls of the closed cross section, as shown in figure (4.18). Portions of the deck slab which do not form the closed section, and appendages to the girder walls (such as stiffeners to steel plates), have negligible influence on the torsional rigidity. Therefore these components can safely be excluded from consideration in the calculation of D_{xy}.

* n_s is the ratio of shear moduli for steel and concrete and it is equal to 1.0 for the case of all concrete construction and 0.88 n for steel-concrete construction (n is the ratio of the moduli of elasticity of steel and concrete).

* G_c is the shear modulus of the concrete.

$$G_c = \frac{E_c}{2(1 + \nu_c)} \tag{4.26}$$

where ν_c is Poisson's ratio for concrete.

* $\oint ds/t'$ is the contour integral along the meridian line of the reciprocal of the wall thickness.

4.5.4 Diaphragms

The second analysis is conducted for the influence of diaphragms on the natural frequency and on the change in mode type. Different types of diaphragms can be used for this purpose, however, we limited our study in this thesis on solid plate diaphragms only.

These plates were provided at both ends of the bridge within each box (that makes a total of four diaphragms per bridge). The nine-node shell element is also used to model the diaphragms and the material modelling is the same as for the girders.

The analysis was carried out for both free vibration and forced vibration responses.

4.5.5 Bracing Systems

In a third analysis, some bracing systems were provided between boxes to analyze their influence on the natural frequency of the bridge and on the change in the first mode shape. The number of these bracing systems was increased from one to five. The first one was exactly in the middle of the bridge. The second case, one bracing at each support. One bracing was added at the middle for the third case and finally two others were placed one quarter of the length (of the bridge) from the ends.
In this thesis, the bracing systems used have an X-shape and are modelled using an isoparametric beam element having six degrees of freedom.
The analysis was carried out for both free vibration and forced vibration responses.

4.6 Forced Vibration

The forced vibration study was carried out for different parameters.
The first analysis was the influence of the diaphragms on the peak acceleration. A second analysis was carried out to see the influence of bracing system on the peak acceleration. The last analysis was the response of the bridge to different speeds of the truck.
For comparison, the bridge was loaded in two ways (see Fig 4.16):
-Truck in the center lane.
-Truck in the middle of the bridge.

4.7 Approximations

For the finite element idealization of the bridge, the top flanges were included in the thickness of the concrete deck slab. This was performed by converting the steel top flanges into concrete unit. Another approximation was made when applying the loads. The actual loads were replaced by the reactions to simplify the finite element mesh.

	ADINA			Beam Theory	
	Frequency (rad/sec)	Frequency (cycles/sec)	Period (sec)	Frequency (rad/sec)	Error
mode 1	23.46	3.734	0.2678	23.51	0.21%
mode 2	93.20	14.83	0.06741	94.05	0.90%
mode 3	207.4	33.01	0.03030	211.61	1.98%
mode 4	363.2	57.81	0.0173	376.19	3.45%

Table 4.1: Comparison between ADINA and Beam theory (Using beam elements)

		ADINA			Beam Theory	
		Frequency (rad/sec)	Frequency (cycles/sec)	Period (sec)	Frequency (cycles/sec)	Error
mode 1	(F)	22.80	3.629	0.2755	3.734	2.81%
mode 2	(T)	29.09	4.631	0.216	–	–
mode 3	(F)	63.95	10.18	0.09825	14.83	31.35%

Table 4.2: Comparison between ADINA and Beam theory (Using shell elements)

1 2

[1](T) : Torsional mode.

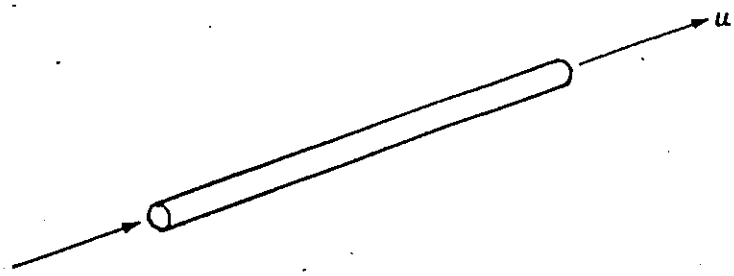
[2](F) : Flexural mode.

	ADINA			Finite Strip	
	Frequency (rad/sec)	Frequency (cycles/sec)	Period (sec)	Frequency (rad/sec)	Error
mode 1	0.003196	0.0005086	1966.0	0.003264	2.08%
mode 2	0.003596	0.0005726	1746.0	0.004070	11.65%
mode 3	0.006157	0.0009798	1021.0	0.007958	22.63%

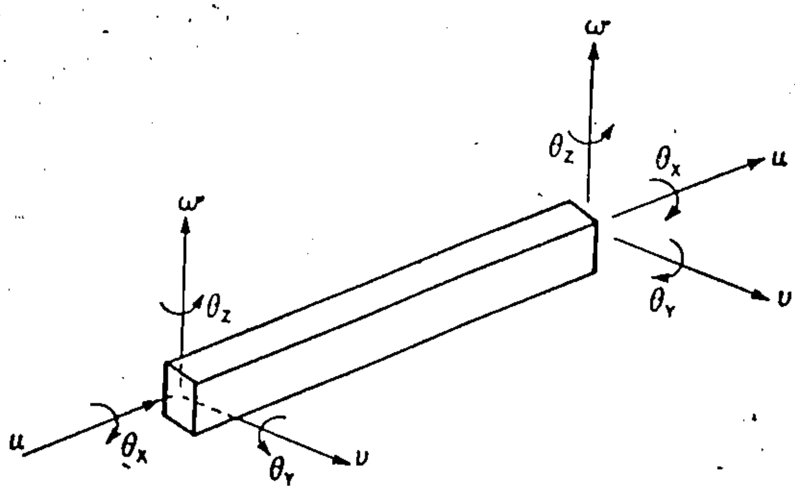
Table 4.3: Comparison between ADINA and Finite Strip Method

Material Constants	Concrete	Steel
E (N/m^2)	$2.65 * 10^{10}$	$2.0 * 10^{11}$
ν	0.15	0.30
ρ (Kg/m^3)	2400	7800

Table 4.4: Material properties for concrete and steel

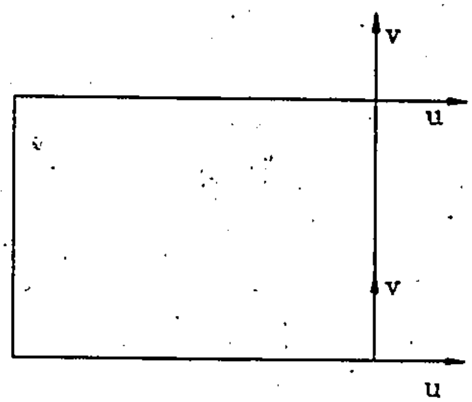


a) Bar Element (One Degree of Freedom at Each Node).

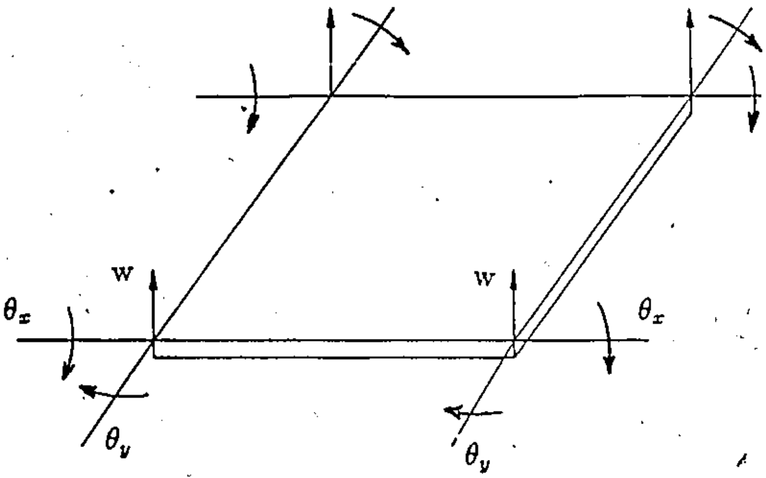


b) Beam Element (Six degrees of Freedom at Each Node).

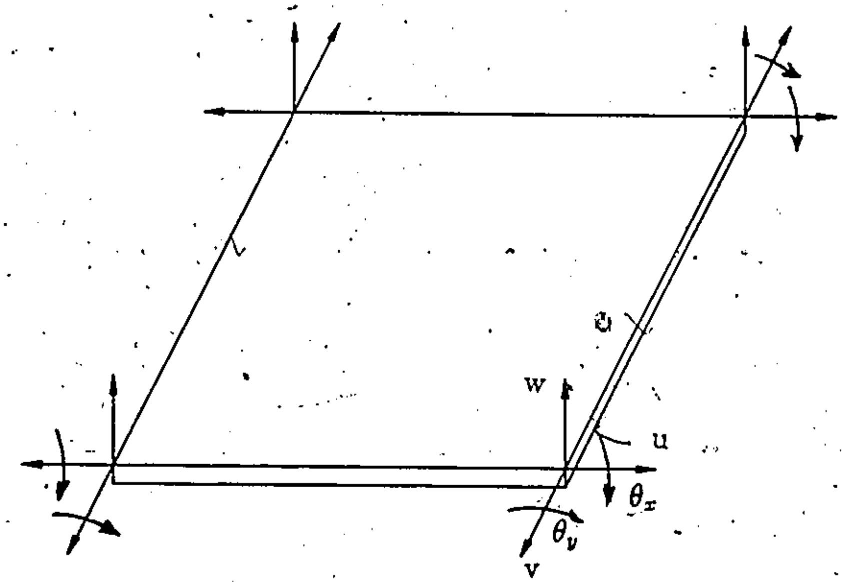
Figure 4.1: One-Dimensional Elements.



a) Membrane Element (Two Degrees of Freedom per Node).



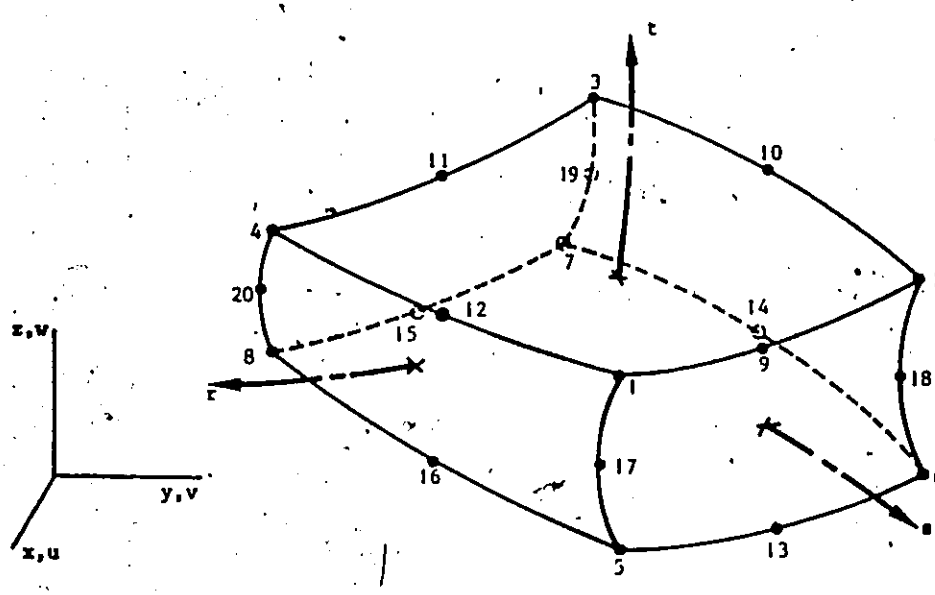
b) Flexural Element (Three Degrees of Freedom per Node).



c) Membrane-Flexural Element (Five Degrees of Freedom per Node).

Figure 4.2: Some Typical Rectangular Elements

Tetrahedral Elements.



Twenty-Node Hexahedral Elements.

Figure 4.3: Some Typical 3-D Elements.

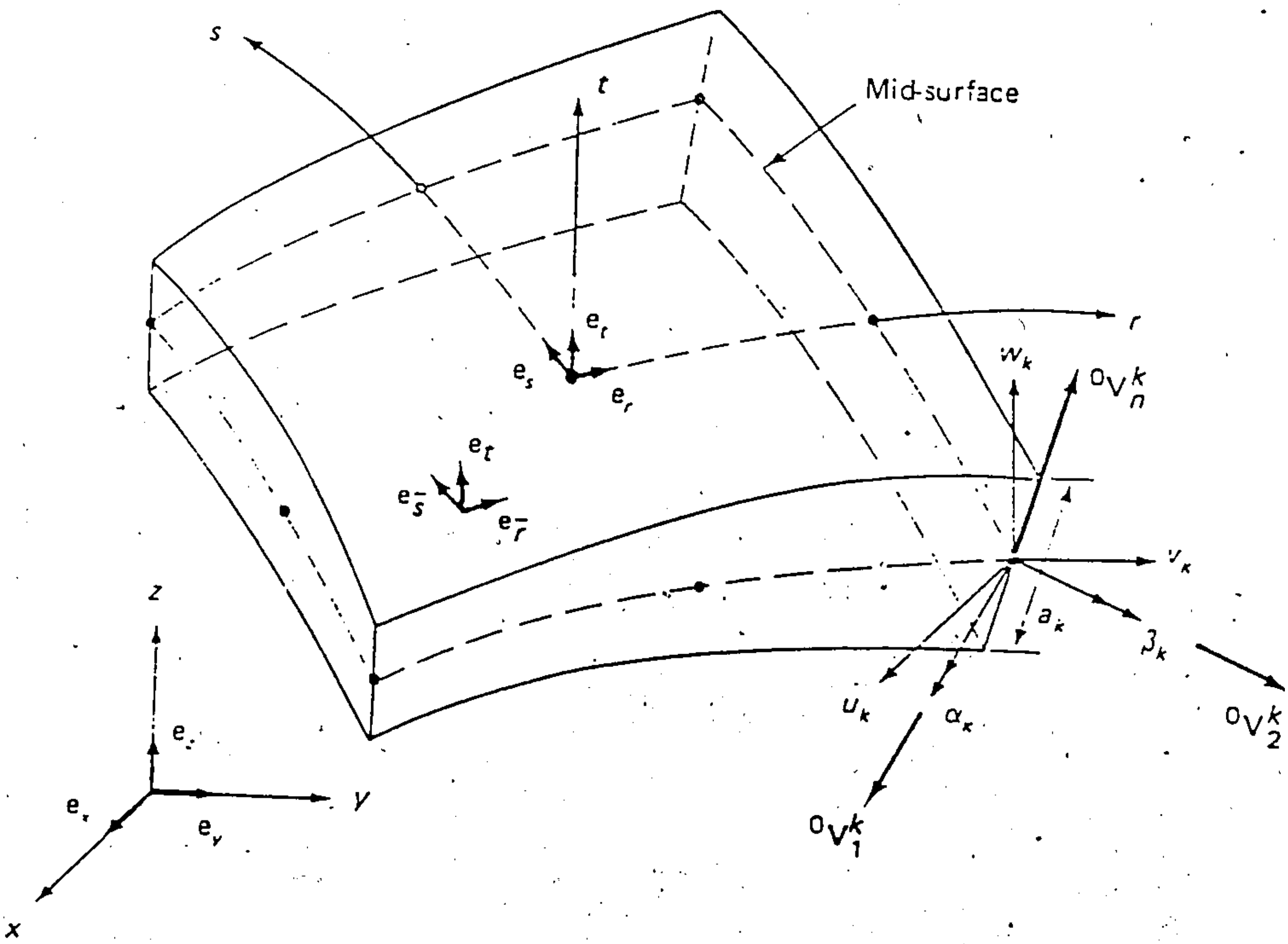
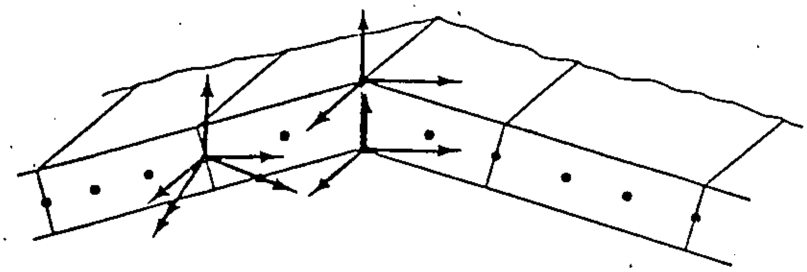
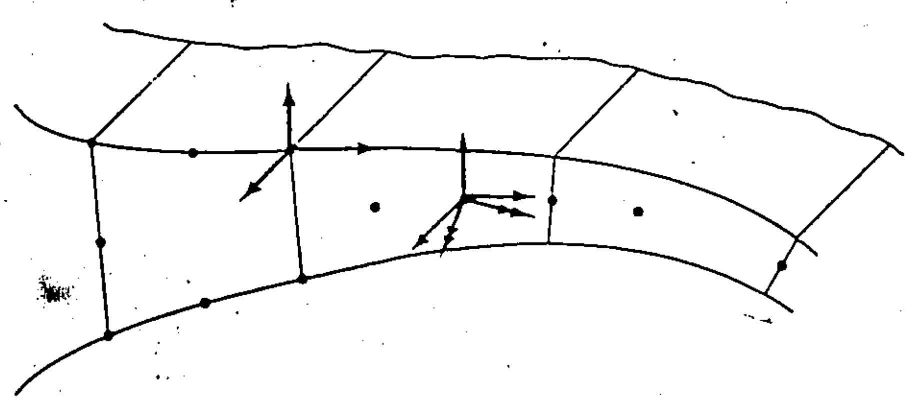


Figure 4.4: Nine-Node Shell Element.

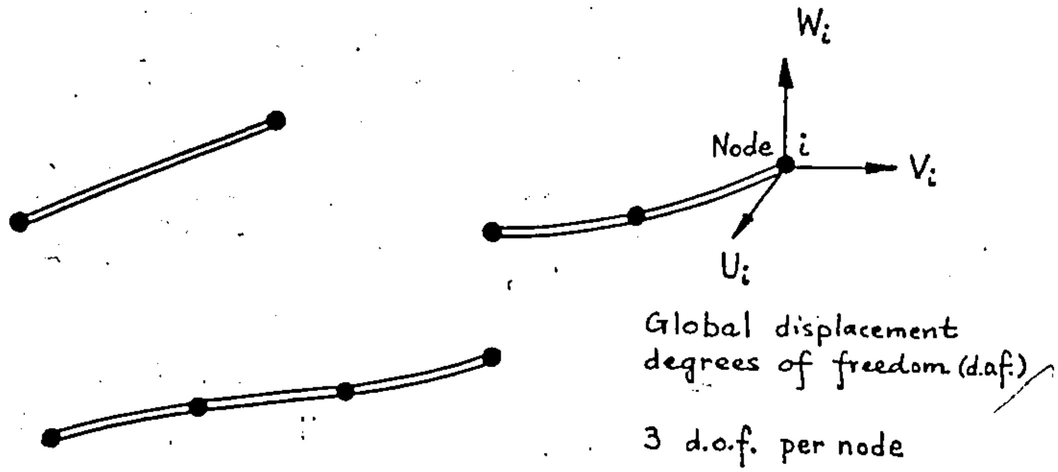


a) Shell Intersections.



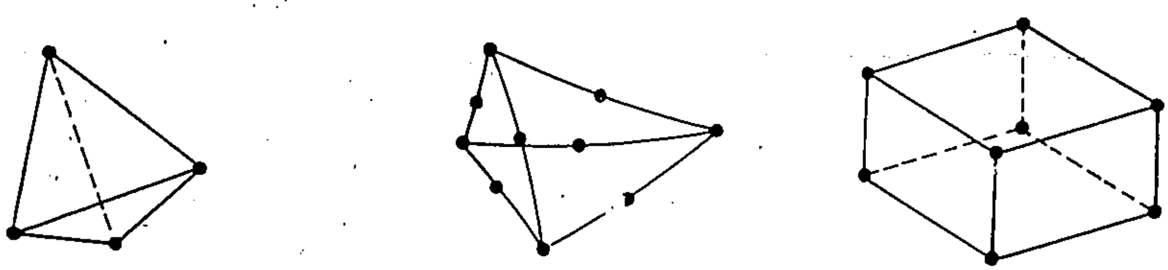
b) Solid to Shell Intersection.

Figure 4.5: Use of Shell Transition Elements.



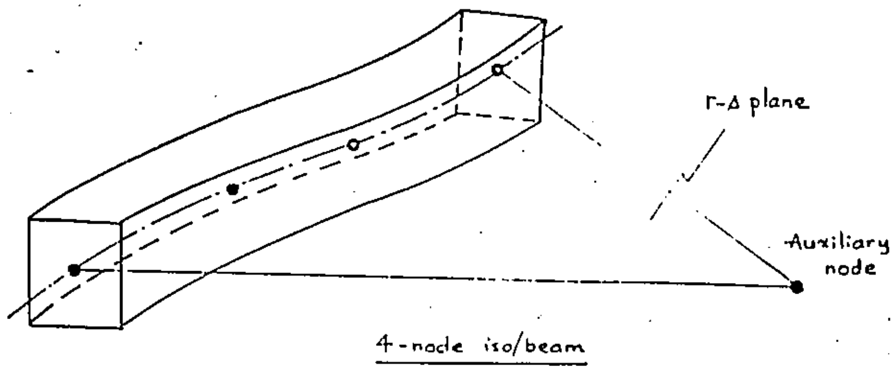
a) Truss Elements.

b) Some 2-D Elements.

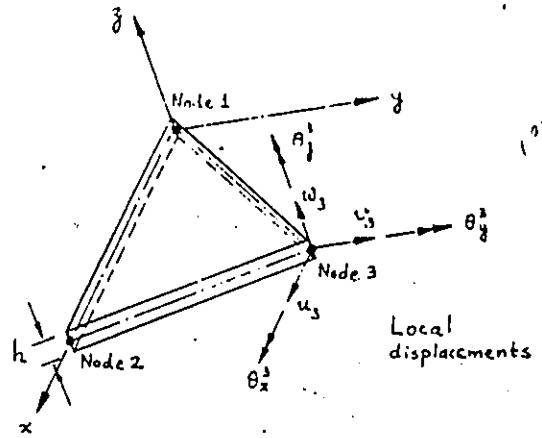


c) Some 3-D Elements.

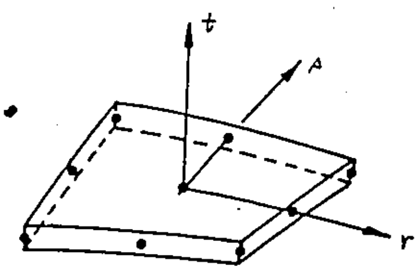
Figure 4.6: Some Typical Elements Used in ADINA.



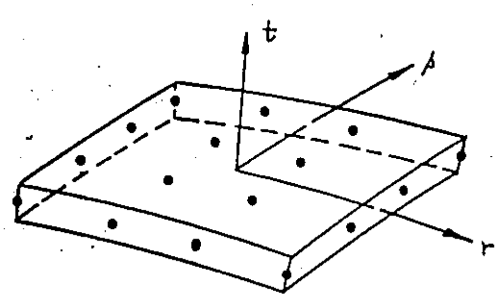
d) 4–Nodes Isoparametric Beam Element.



e) Triangular Plate Element.

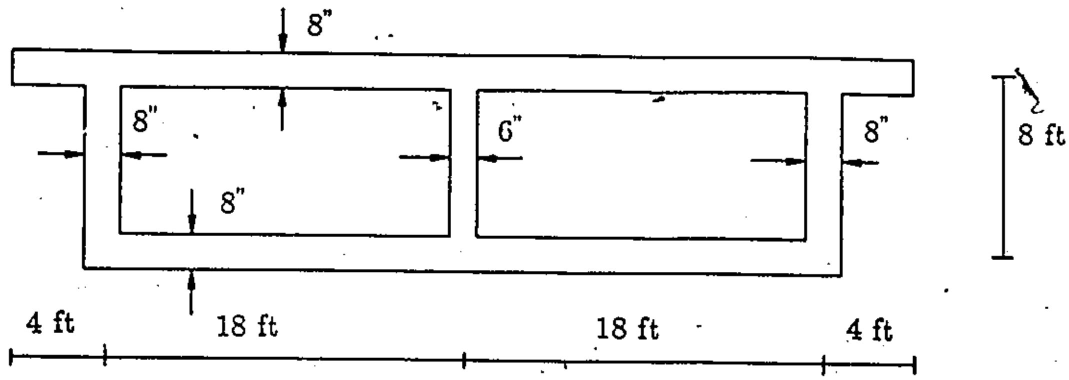


f) 9–Nodes Shell Element.

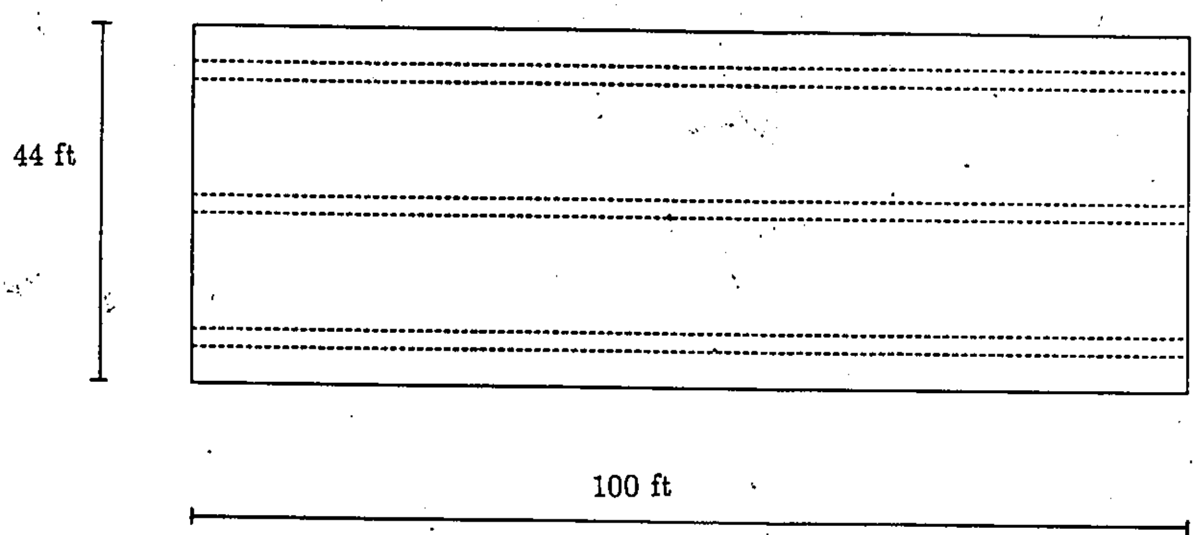


g) 16–Nodes Shell Element.

Figure 4.6: (Continued) Some Typical Elements Used in ADINA.

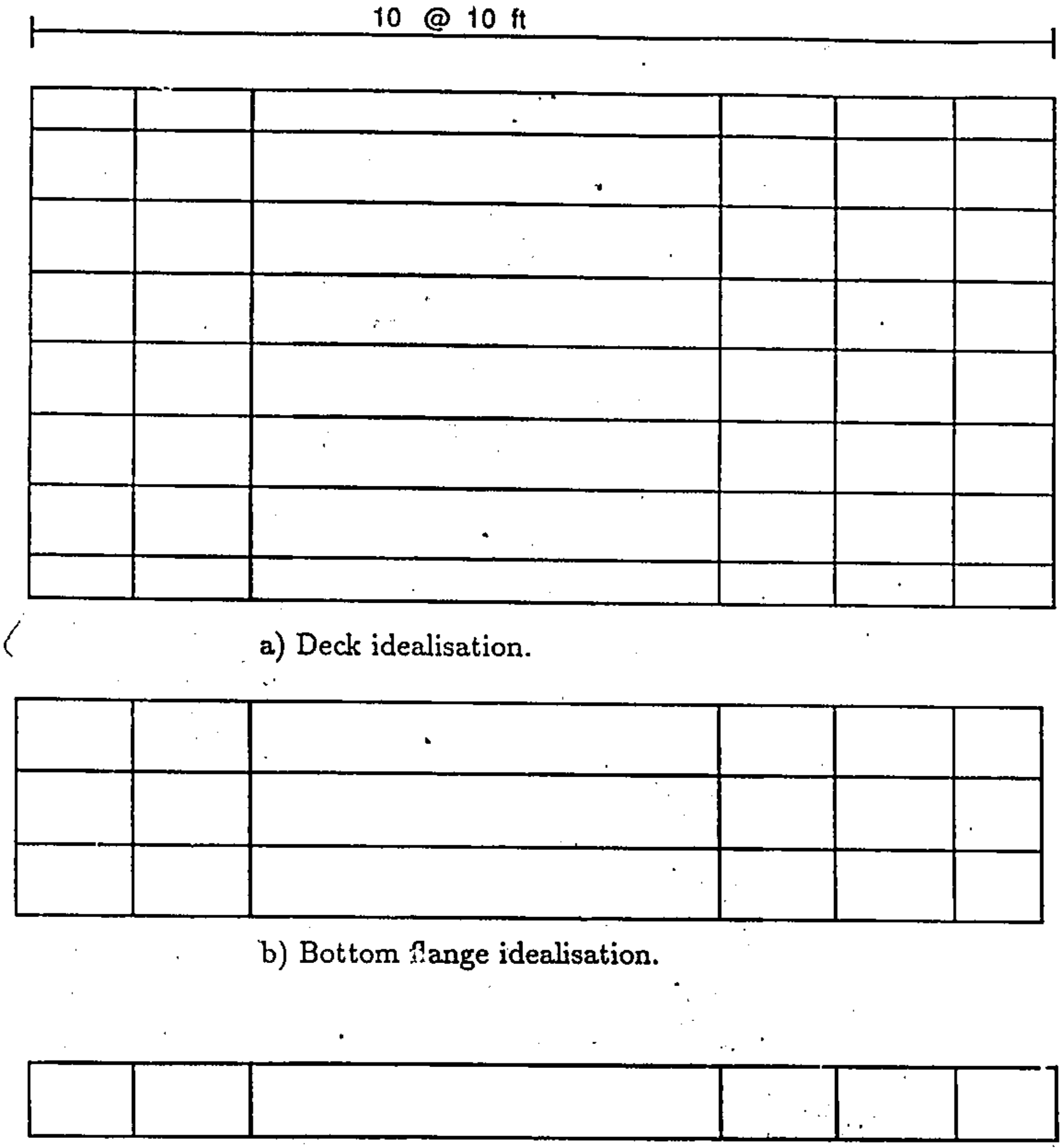


a) Section of the box girder bridge.



b) Plan of a straight box girder bridge.

Figure 4.7: Details of the Box Girder Bridge (Used for Finite Strip Analysis).

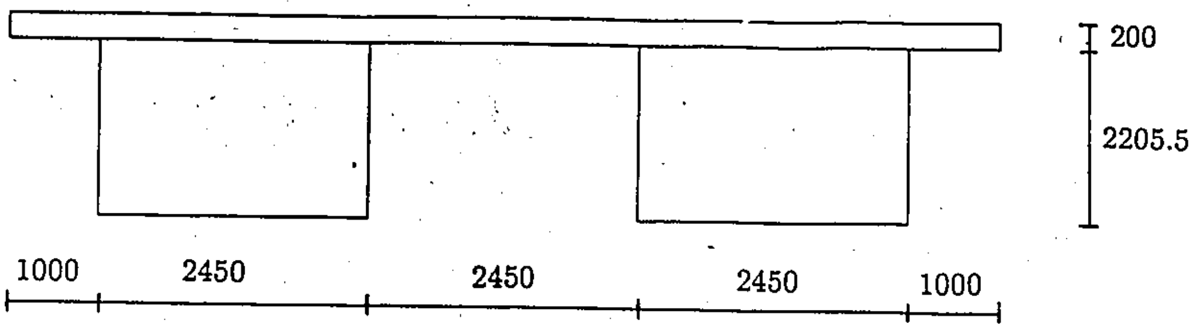


a) Deck idealisation.

b) Bottom flange idealisation.

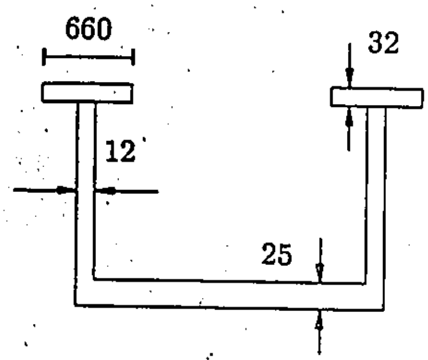
c) Web idealisation.

Figure 4.8: Idealization of the Box Girder Bridge (To Compare With Finite Strip Method).

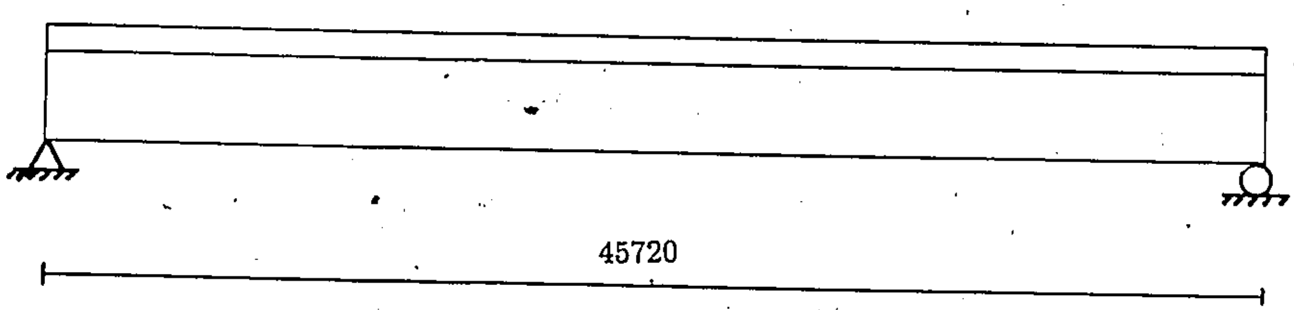


a) Cross Section

All dimensions in mm.

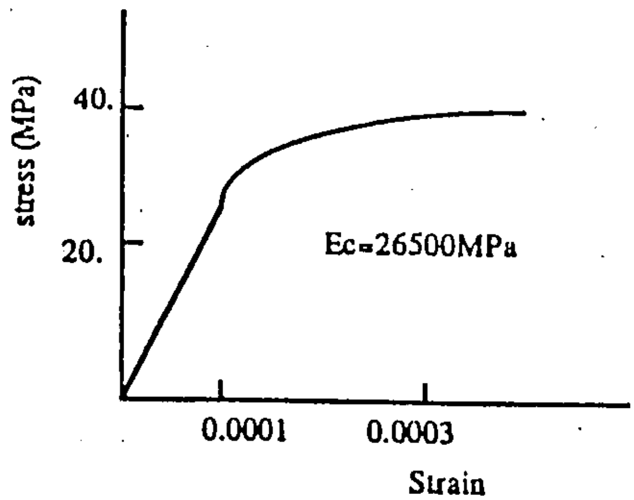


b) Details of the Box

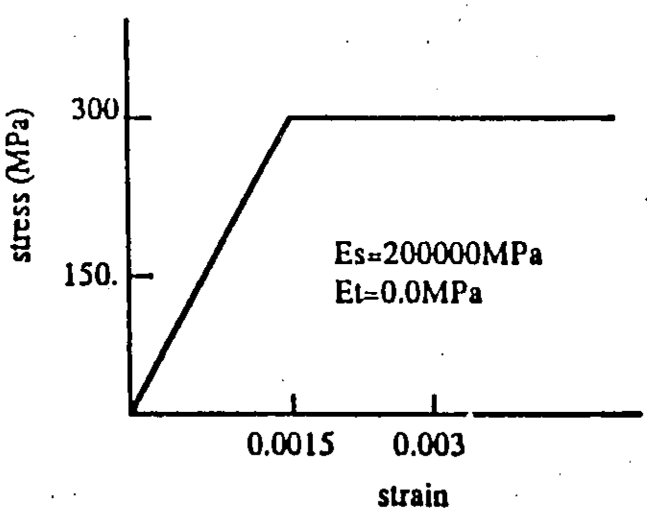


c) Elevation

Figure 4.9: Cross Section and Elevation of the Bridge.

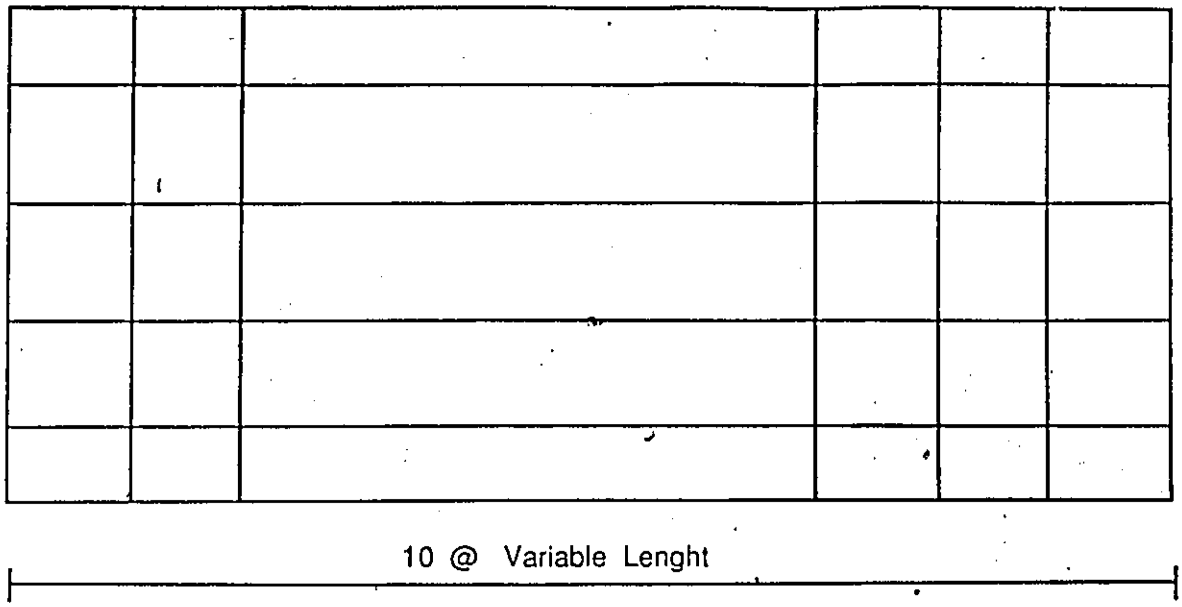


a) Concrete



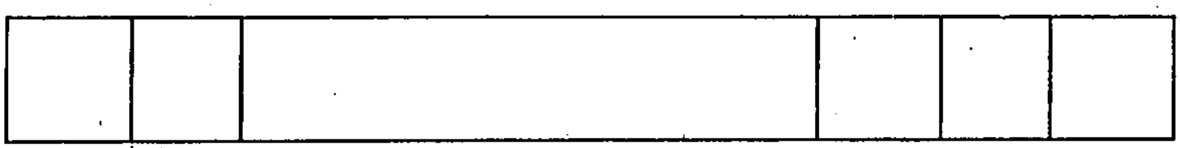
b) Steel

Figure 4.10: Stress–Strain Relationship



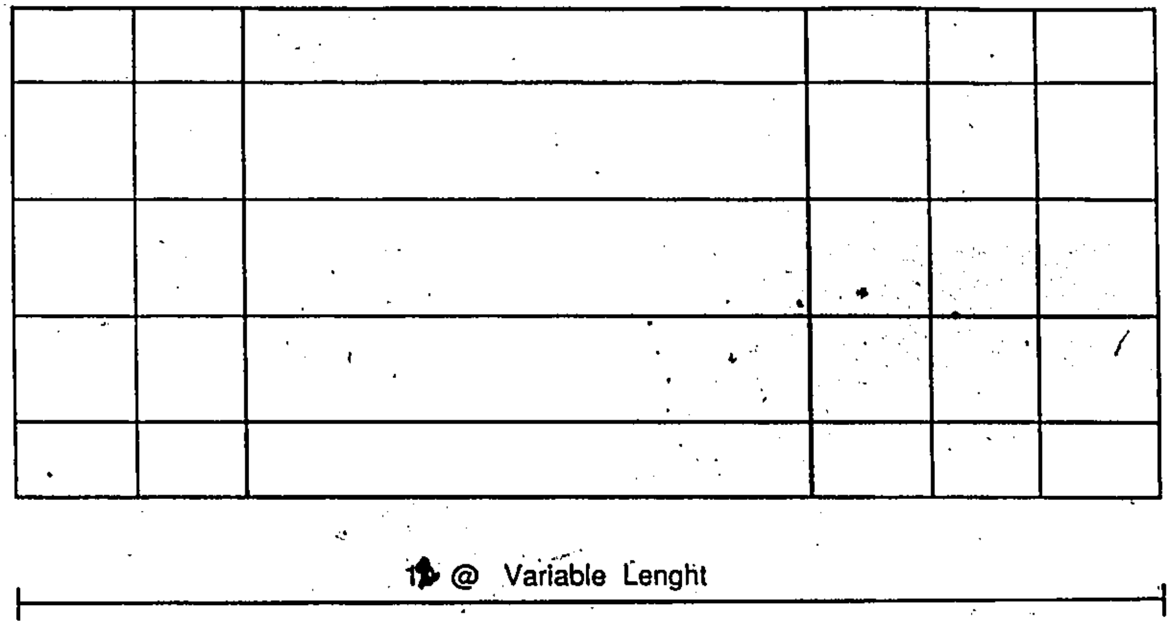
a) Deck idealisation.

b) Bottom flange idealisation.

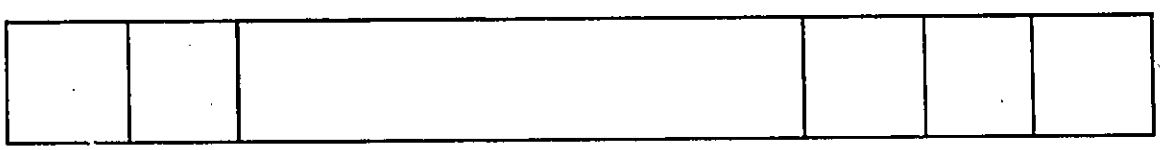


c) Web idealisation.

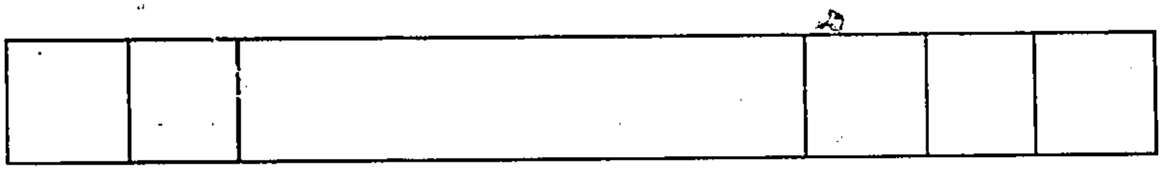
Figure 4.11: First Mesh Used (110 elements and 445 nodes).



a) Deck idealisation.

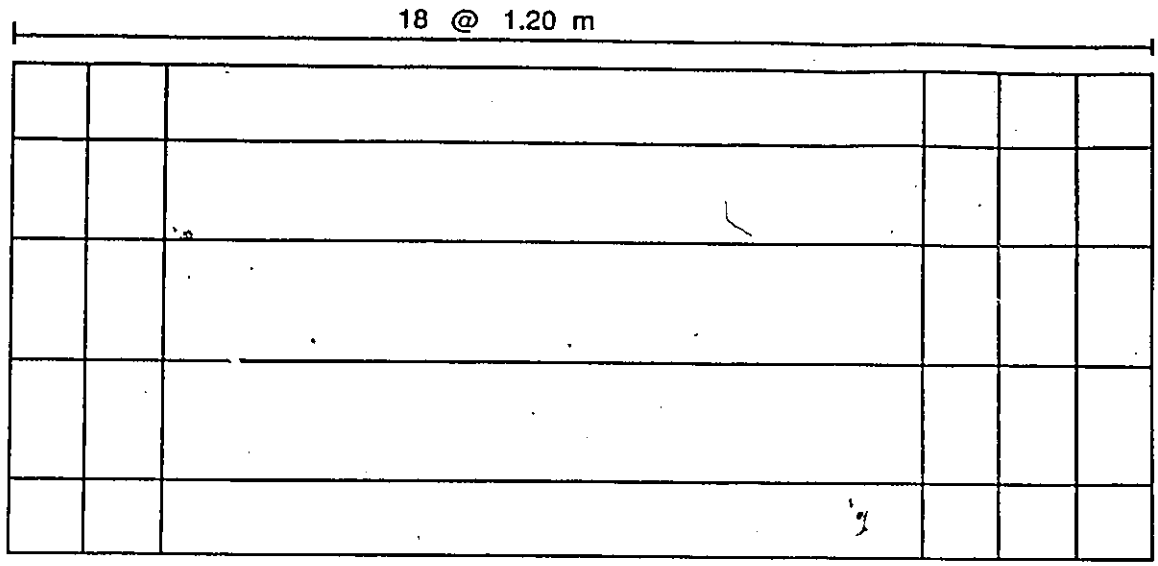


b) Bottom flange idealisation.

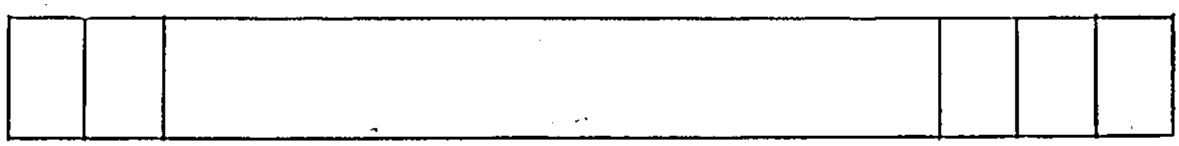


c) Web idealisation.

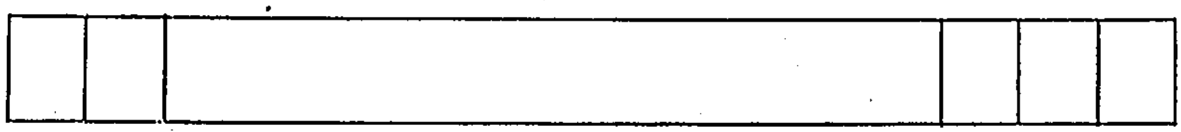
Figure 4.12: Second Mesh Used (132 elements and 525 nodes).



a) Deck idealisation.



b) Bottom flange idealisation.



c) Web idealisation.

Figure 4.13: Mesh Used for Forced Vibration Analysis (198 Elements and 777 No des.)

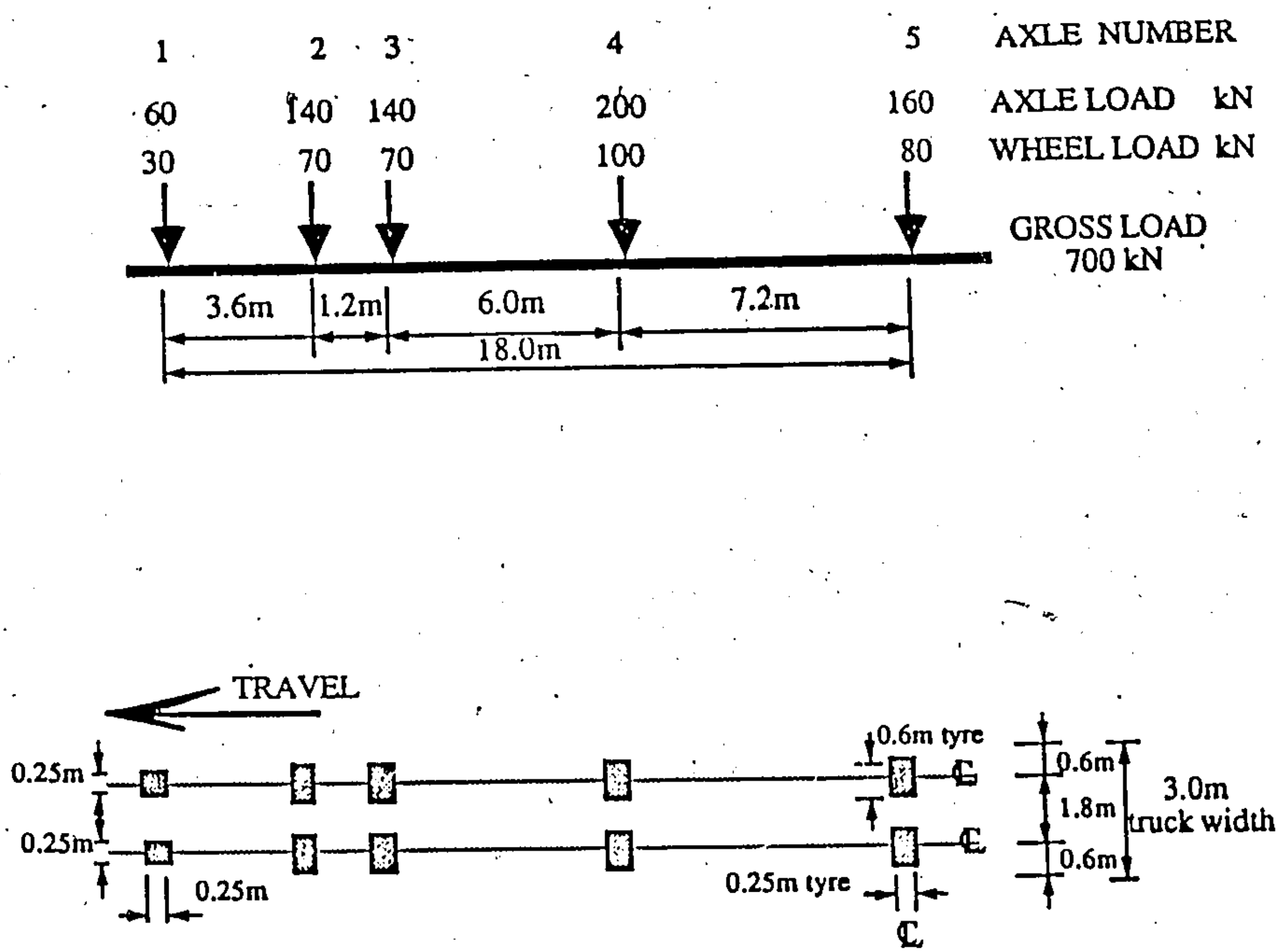
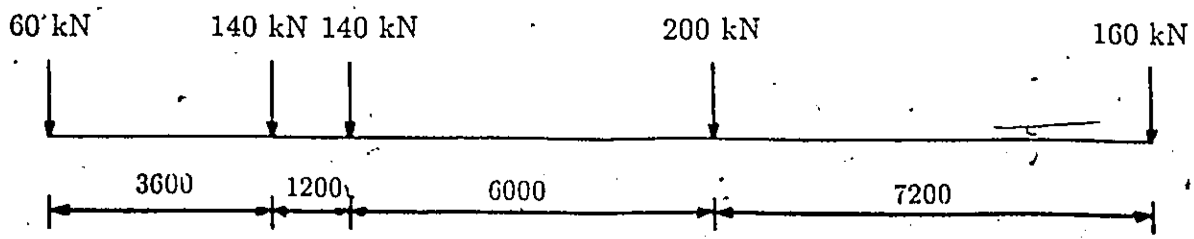
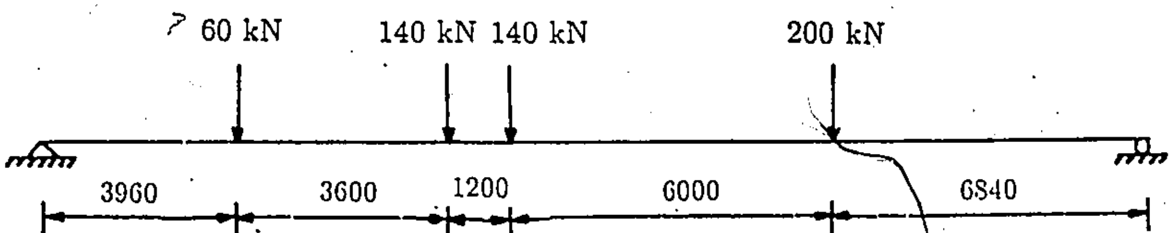


Figure 4.14: OHBD Loading Truck.

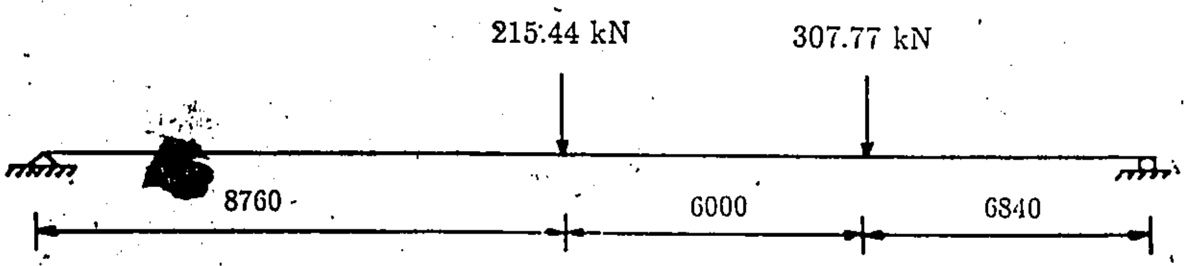


All dimensions in mm.

a) OHBD truck load



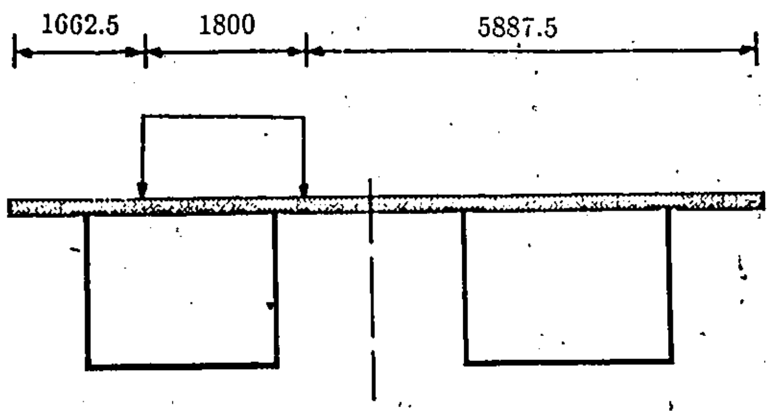
b) Position of the load creating maximum moment.



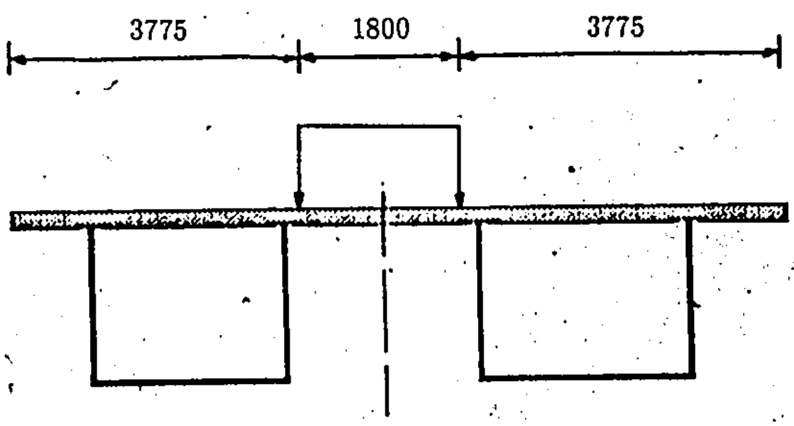
c) Actual load creating maximum moment.

Figure 4.15: Simplification of the Load for Longitudinal Position.

All dimensions in mm.



a) Truck in the middle of the lane.



b) Truck in the middle of the bridge.

Figure 4.16: Transverse Position of the Truck.

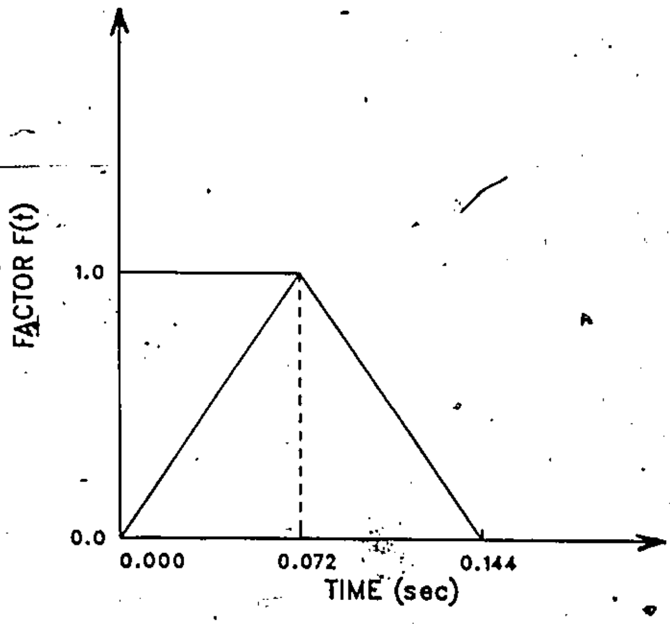


Figure 4.17: Time Function.

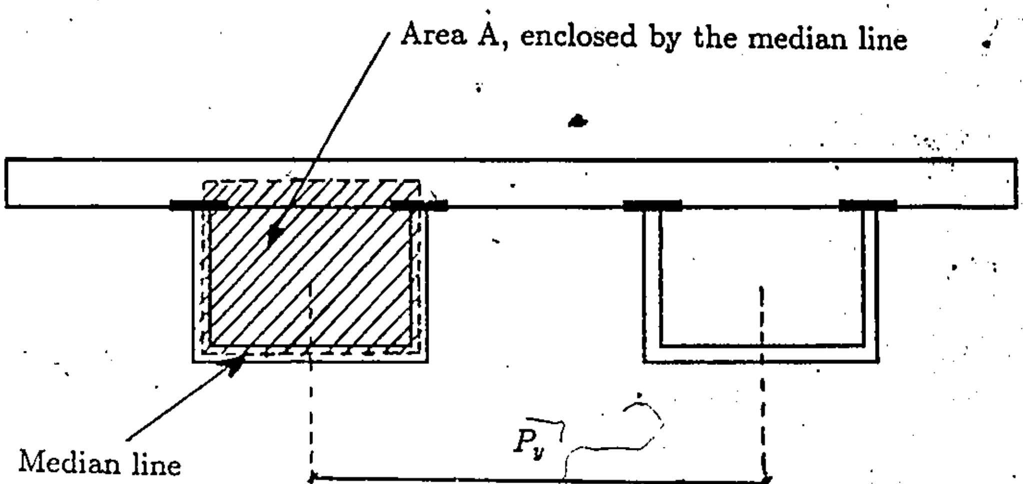


Figure 4.18: Cross Section of a Multispine Bridge.

Chapter 5

RESULTS AND DISCUSSIONS

5.1 General

As stated in the previous chapter, the main parameters were investigated in this thesis to study their effect on the change in the governing mode type. The selected parameters were categorized according to type of vibration as follows:

FREE VIBRATION :

- - Aspect ratio (R).

- - Rigidity ratio (D_x/D_{xy}).
- - Diaphragms within boxes.
- - X-Bracings between boxes.

FORCED VIBRATION :

- - Speed of moving load (Truck).
- - Diaphragms within boxes.
- - X-bracings between boxes.

The range of these parameters was selected based on the change of the first mode type (change from torsional to flexural or vice versa. An example is shown in Fig. 5.1). And this range varies from one parameter to another.

For the free vibration study, the finite element program produces the values of the natural frequencies and the corresponding mode shapes at nodal points. This work is limited to the variation of the first natural frequency with these parameters.

For the forced vibration response the output includes the values of displacements, velocities and accelerations at nodal points for each time step. However, because of the massive data output and the fact that peak accelerations were found to occur at the bridge mid-section the output was limited to this critical section.

In general these parameters affect the first natural frequency of the box girder bridge and especially the first mode type.

In this chapter we will present the results of ten separate analyses which are as follows :

A) Free Vibration Analysis:

1. Aspect ratio effect on the first mode type.
2. The effect of the rigidity ratio (Flexural rigidity over torsional rigidity) on the first mode type.
3. The effect of change in interior diaphragm thicknesses on the first mode type.
4. The effect of the number of X-bracing on the first mode type. Unlike the interior diaphragms, these bracings were placed between the boxes.

B). Forced Vibration Analysis:

This analysis was performed for two distinct load cases with respect to the transverse location of the OHBD truck. In one load case, the truck was located in accordance with OHBD code in the center of a lane, whereas in the second load case the truck was positioned concentrically with bridge transverse centerline. In this way comparison between transversely eccentric and concentric truck loading can be studied.

The resulting different analyses performed were:

1. The effect of the change of the truck's speed on the peak accelerations with truck placed in the center of the lane.
2. The effect of the change of the truck's speed on the peak accelerations with truck placed in the middle of the bridge.
3. The effect of change in diaphragm thicknesses on the peak accelerations with truck placed in the center of the lane.
4. The effect of change in diaphragm thicknesses on the peak accelerations with truck placed in the middle of the bridge.
5. The effect of the number of X-bracing on the peak accelerations with truck placed in the center of the lane.
6. The effect of the number of X-bracing on the peak accelerations with truck placed in the middle of the bridge.

5.2 Free Vibration Analysis

The free vibration analysis was carried out for many bridges, all of which were simply supported twin box-girder bridges.

The analyses were conducted for the following parameters :

5.2.1 Effect of Aspect Ratio

The aspect ratio is defined as the ratio of the length of the bridge to its width. In this study the analysis was conducted on bridges having the same cross-section but different aspect ratios.
The cross section of the simply supported box-girder bridges consists of two steel box girders and a uniform concrete deck with a flexural rigidity (D_x) of $13.34 * 10^9 (N.m)$ and a torsional rigidity (D_{xy}) of $4.645 * 10^9 (N.m)$. The width of the bridges was kept constant (9.35 m) and only the length was changed.

Figure (5.2) shows the variation of the first natural frequency with the aspect ratio for the first type boundary conditions (which corresponds to the pin-roller). It can be seen from this figure that the first natural frequency decreases when the ratio increases. That means, longer bridges tend to have lower frequencies. This also confirms that the natural frequency is related to the inverse of the mass.

Also, for shorter bridges (Aspect ratio between 1 and 2.353) the dominant mode shape is torsional and the second mode is flexural. While for longer bridges (Aspect ratio of 2.44 and up) the dominant mode is flexural.

It is evident from this diagram that the aspect ratio of the bridge plays an important role in determining the nature of the first mode type of the bridge free vibration. The change from torsional mode type to flexural mode seems to take place at bridge aspect ratio of 2.4 corresponding to a first natural frequency of 10.12 Hz.

From table (5.1) it can be seen that for a ratio around the value 2.35 the first and second natural frequencies are very close whereas for aspect ratios far from this value the difference is considerable.

The same analysis was conducted for the second type boundary conditions (which corresponds to the pin-pin). Figure (5.3) shows that the type of curve obtained is the same as the first one except that the critical aspect ratio at which the first mode type changes from torsional to flexural is changed to about 3.1. At this aspect ratio the first natural frequency is about 9.6 Hz. So this change in boundary conditions does not affect tremendously the type of variation of the first frequency with the aspect ratio.

Finally, we can concluded that for this typical cross-section of a twin box girder bridge ($D_x = 13.34 * 10^9\ N.m\ and\ D_{xy} = 4.645 * 10^9\ N.m$) the first mode type changes from torsional to flexural with change in aspect ratio. The values of R=2.40 can be taken as the critical value for the first type boundary and R=3.1 can be taken as the critical value for the second type boundary.

5.2.2 Effect of Rigidity Ratio

The rigidity ratio is defined as the ratio of the flexural rigidity to the torsional rigidity. Figure (5.4) shows the variation of the first natural frequency and the associated natural mode of vibration with the rigidity ratio. The analysis was conducted on a twin box girder bridge with an aspect ratio of 2.353. For the variation of rigidities only the boxes' dimensions were changed while the thickness of the concrete deck remained constant. It can be seen from the figure that the graph can be divided into three distinct regions:

* **Region I** : This region corresponds to the variation of rigidity ratio from zero to 2.25. The first mode type of a bridge having a rigidity ratio less than 2.25 is flexural as shown in the figure.

* **Region II** : Where the rigidity ratio is between 2.25 and 2.50. The first mode type in this region can be either flexural or torsional.

* **Region III** : Defined by a rigidity ratio equal to 2.5 and up. In this region the first mode type of the bridge seems to be only torsional.

As such, for a specific aspect ratio, the first mode type of vibration can change from flexural mode to torsional by increasing the rigidity ratio. The value which limit the central region where either type of modes can occur, depends on the aspect ratio and most probably on some other parameters.

5.2.3 Effect of Diaphragms

In this thesis the diaphragms were used only at the ends of each box and within the boxes. The study of the effect of diaphragm thicknesses on the first natural frequency was carried out for aspect ratios between 1.0 and 4.0. This range corresponds to the case where the first mode is either flexural or torsional.

*** Ratio R=1.0**

At this aspect ratio the bridge is short and square. In figure (5.5) we plotted the variation of the first two natural frequencies for the square box girder bridge without any diaphragms and with diaphragms at the supports with different thicknesses.

It can be seen from this figure that the first mode is always torsional mode regardless of the diaphragms. But the increase in the first natural frequency due to the increase in the thickness of the diaphragms is considerable. Whereas for the second mode, which is of flexural nature, the frequency was almost independent of diaphragm thickness.

For example the increase of the first natural frequency of this square bridge due to 10 mm diaphragms at the supports is equal to 77 % whereas for the second frequency the increase is only 16 %.

*** Ratio R=2.0**

In figure (5.6) the variation of the first two natural frequencies are also plotted against the variation of the diaphragm thickness.
Initially the first mode was torsional, and by adding the diaphragms the mode type changes to flexural.
The two curves corresponding to each mode type have a common point around the value t=2.0 mm which corresponds to the limit of dominant torsional mode and dominant flexural mode. Note that the first mode is always the one with the smallest natural frequency.
The increase of the first natural frequency due to a 10 mm diaphragms is 28.0 % while for the second the increase is 36.0 %.

*** Ratio R=2.353**

This point is very close to the critical ratio for the change of the first mode type from torsional to flexural. Figure (5.7) shows the variation of the two first natural frequencies with the variation of the thickness of the diaphragm.
Initially the first mode was torsional, but as soon as a 1 mm diaphragm was added at the supports, the first mode changed to flexural. The intersection of the two curves is around the value t=0.35 mm.
From a practical point of view the first mode type can always be described

as flexural since the diaphragm thickness is usually around 10 mm.
The first natural frequency was increased by 8.0 % when placing a 10 mm diaphragm at each support and the second natural frequency was increased by 55.0 %.

*** Ratio R=4.0**

In Figure (5.8), the variation of the natural frequencies was plotted against the diaphragm thickness. Without any diaphragm the first mode type was observed to be of flexural type.
It can be seen from the figure that the provision of diaphragms at each support did not have any appreciable effect on the first natural frequency of vibration. The increase of this frequency was very small (less than 1 %) while the increase for the second frequency was 32.0 %.

The influence of diaphragms are summarized in Table (5.2) and the values are plotted in Figure (5.9). It can be concluded from the figure that the provision of diaphragms affects the first mode only when it is a torsional mode. And the variation of the natural frequency due to the diaphragms could be very important depending on the aspect ratio.

5.2.4 Effect of Bracings

A similar study to the previous one was carried out to see the influence of bracings on the natural frequency of bridge free vibration. The bracings used have an X-shape consisting of a rectangular cross section of 5 by 10 cm. The analysis was conducted for the following bracing configurations :

- - One bracing at the mid-span.
- -One bracing at each support (total of two).
- - One bracing at the mid-span and two bracings at the supports (total of three).
- - One bracing at the mid-span, two at the supports and two at quarter-span (total of five).

These bracings were placed without any diaphragms or other bracing systems within boxes. Figures (5.10) to (5.13) show the variation of the two first natural frequencies with the different bracing configurations used for the four aspect ratios mentioned previously.

For an aspect ratio of 1.0 (see Figure 5.10), without any bracing the first mode is torsional. With the provision of three bracings and up the first mode becomes flexural.
It seems that the value of three bracings is a critical value where the first mode type changes from torsional to flexural. There is (29 %) increase in

the first natural frequency when the number of X-bracing is increasing from one at the bridge mid-span to three X-bracings at supports and mid-span. With provision of two more X-bracings (total of five) at a quarter of span the increase is very small. In the second mode the frequency decreases with increase in number of cross-bracings and the decrease due to the presence of five bracing systems between boxes is equal to 32 %.

Figure (5.11) shows the variation of frequencies with the number of cross-bracings used for an aspect ratio R=2.0. The two curves corresponding to flexural and torsional modes have a common point around the value 2 in the X axis which indicates the limit in change of first mode type.
There is about 23 % increase in first natural frequency when increasing the number of X-bracing from one to five. While the second natural frequency increases slightly with the number of bracing (3.0 % increase). Note that the first mode has always the lowest natural frequency.

The frequency variation for aspect ratio R=2.353 is shown in Figure (5.12). Again the variation of first and second natural frequencies vibration are the same as for the previous aspect ratio (R=2.0) except that the common point which limit the flexural mode from the torsional is now around one . The increase in first frequency due to five bracing systems is 5 % while the increase in second frequency is 20 %.

Figure (5.13) shows the variation of frequencies for aspect ratio R=4.0. Initially, and without the presence of any bracing, the first mode of vibration was flexural. So, from the figure it can be seen that the first frequency is almost unchanged (decrease of -0.2 %) while the second frequency increases with the number of bracings (increase of 19 %).

Finally, the presence of bracing systems between boxes can affect the first natural frequency if the first mode type is torsional. The values of Table (5.3) are plotted in Figure (5.14) which summarize the variation of first frequency of vibration with the number of bracings.

5.3 Forced Vibration Analysis

With the recommendations of the OHBDC, the truck load should be positioned at the center of a lane. In addition to this, we conducted another analysis of bridge response where the truck load is in the middle of the bridge, for comparison purpose. In both positions, the analysis was carried out to study the effect of the truck's speed on the response and the influence of interior diaphragms and exterior X-bracings on the peak accelerations. The bridge analyzed was a simply supported twin box-girder with aspect ratio of 2.31 (that is a length of 21.6 m and a width of 9.35 m) and a rigidity ratio of 2.86.
A length of time of four seconds was retained for all the analyses since at that time, and for any speed between 20 and 60 Km/h the truck should have cleared the bridge.

5.3.1 Speed of the Truck

Appendix B contains some of the figures representing the variation of the maximum accelerations at each time step for a truck speed of 60 Km/h and for both load cases : truck in the middle of the bridge and truck in the center of the lane.

These variations are of the same type. All of them are harmonic in nature. In Figure (5.15) we represent the maximum accelerations occurred during the passage of the truck against the speed of the truck. It can be seen that there is a harmonic relationship between the two variables for both cases. Except that when the truck is in the centre of the lane the accelerations are higher than the one occurred when the truck is in the middle of the bridge. For a speed of 60 Km/h the acceleration is the highest one, so we maintain this speed to study the response of the bridge.

5.3.2 Effect of Diaphragms

In this part we studied the effect of diaphragms on the peak accelerations due to the passage of the truck. The diaphragms were placed at each support within the boxes. The truck crossed the bridge at a speed of 60 Km/h.

In Figure (5.16) we plotted the variation of the peak accelerations against the thickness of diaphragms used. As can be seen we can divide the graph into two distinct regions:

-The first region which is limited by the values zero and 3.0 corresponds to the region where the first dominant mode is torsional.

-In the second region, the first mode changed to become flexural for a diaphragm's thickness of 3 mm and up.

It can be seen from the figure that the peak acceleration decreases considerably in the first region with the provision of diaphragms. This decrease corresponds to a value of -41 %. While in the second region, there is almost no change in the peak acceleration when adding the diaphragms. The variation of the accelerations in this region was 2 %.

The same variation was obtained for the second case where the truck is in the middle of the bridge, except that the values of peak accelerations are smaller than the previous one.

Finally, the provision of diaphragms reduced considerably the peak accelerations. Note that initially the first mode type of the bridge studied was torsional. The decrease of the peak acceleration due to a 10 mm diaphragm was 50 % when the truck was in the middle of the bridge and 40 % when the truck was in the center of the lane.

5.3.3 Effect of Bracings

The next analysis was carried out to see the influence of the X-bracing systems on the peak accelerations. In Figure (5.17) we plotted the variation of the peak accelerations against the number of X-bracings provided between the boxes. Again, like the previous analysis, the graph can be divided into two distinct regions:

- The first region is limited by the values zero and 2. It can be seen from the graph that the presence of one cross bracing at the mid-span of the bridge did not affect the peak acceleration. The variation is almost negligible for both load cases (0.36 % when the truck is in the center lane and 1.7% when the truck is in the middle of the bridge).

But as soon as we placed two cross bracings at the supports the variation in the peak acceleration is considerable. For the first load case, when the truck is in the center of a lane the peak acceleration decreases by an amount of 18.2 % while for the second load case, when the truck is in the middle of the bridge the decrease is 12.6 %.

In the second region, which is limited by the values 2 and 5, the peak acceleration is almost constant for both load cases.

Finally, we conclude that the provision of cross bracing can reduce considerably the pea'. acceleration only when the bracing are placed at the supports. Any bracing placed between the supports has a small effect (if not) on the peak acceleration.

Aspect of Ratio	ω (rad/sec)		f (cycles/sec)		T (sec)	
	mode 1	mode 2	mode 1	mode 2	mode 1	mode 2
1.0	98.19 (T)	199.7 (F)	15.63	31.79	0.06399	0.03146
1.3	87.10 (T)	152.1 (F)	13.86	24.21	0.07214	0.04131
1.6	78.48 (T)	117.7 (F)	12.49	18.73	0.08006	0.05339
2.0	69.74 (T)	85.92 (F)	11.10	13.68	0.0901	0.07313
2.31	63.41 (T)	68.25 (F)	10.10	10.86	0.09908	0.09206
2.353	63.68 (T)	66.79 (F)	10.13	10.63	0.09867	0.09407
2.445	62.78 (F)	62.28 (T)	9.991	9.912	0.1001	0.1009
2.8	49.98 (F)	57.07 (T)	7.954	9.083	0.1257	0.1101
3.2	39.66 (F)	52.47 (T)	6.312	8.351	0.1584	0.1197
3.5	33.81 (F)	49.49 (T)	5.381	7.876	0.1858	0.1270
4.0	26.51 (F)	45.15 (T)	4.220	7.186	0.2370	0.1392
4.5	21.30 (F)	41.35 (T)	3.390	6.581	0.2950	0.1229

Table 5.1: Analysis Results For Different Aspect Ratios(First Boundary)

1 2

[1](T) : Torsional mode.

[2](F) : Flexural mode.

Effect of Ratio	Without Diaphragms	Diaphragm's Thickness			
		t= 1mm	t= 3mm	t= 6mm	t= 10mm
1.0	15.63 (T)	17.96 (T)	21.14 (T)	24.47 (T)	27.72 (T)
2.0	11.10 (T)	12.66 (T)	14.00 (F)	14.14 (F)	14.23 (F)
2.353	10.13 (T)	10.70 (F)	10.79 (F)	10.86 (F)	10.91 (F)
4.0	4.220 (F)	4.225 (F)	4.232 (F)	4.238 (F)	4.243 (F)

Table 5.2: Efect of Diaphragm's Thicknesses on First Mode Type (For Different Aspect Ratios)

Effect of Ratio	Without Bracing	Total Number of Bracing (N)			
		1	2	3	5
1.0	15.63 (T)	17.35 (T)	20.53 (T)	21.29 (T)	21.38 (F)
2.0	11.10 (T)	11.20 (T)	13.74 (F)	13.71 (F)	13.68 (F)
2.353	10.13 (T)	10.15 (T)	10.66 (F)	10.64 (F)	10.62 (F)
4.0	4.220 (F)	4.215 (F)	4.222 (F)	4.216 (F)	4.211 (F)

Table 5.3: Effect of the Number of Bracing on First Mode Type (For Different Aspect Ratios)

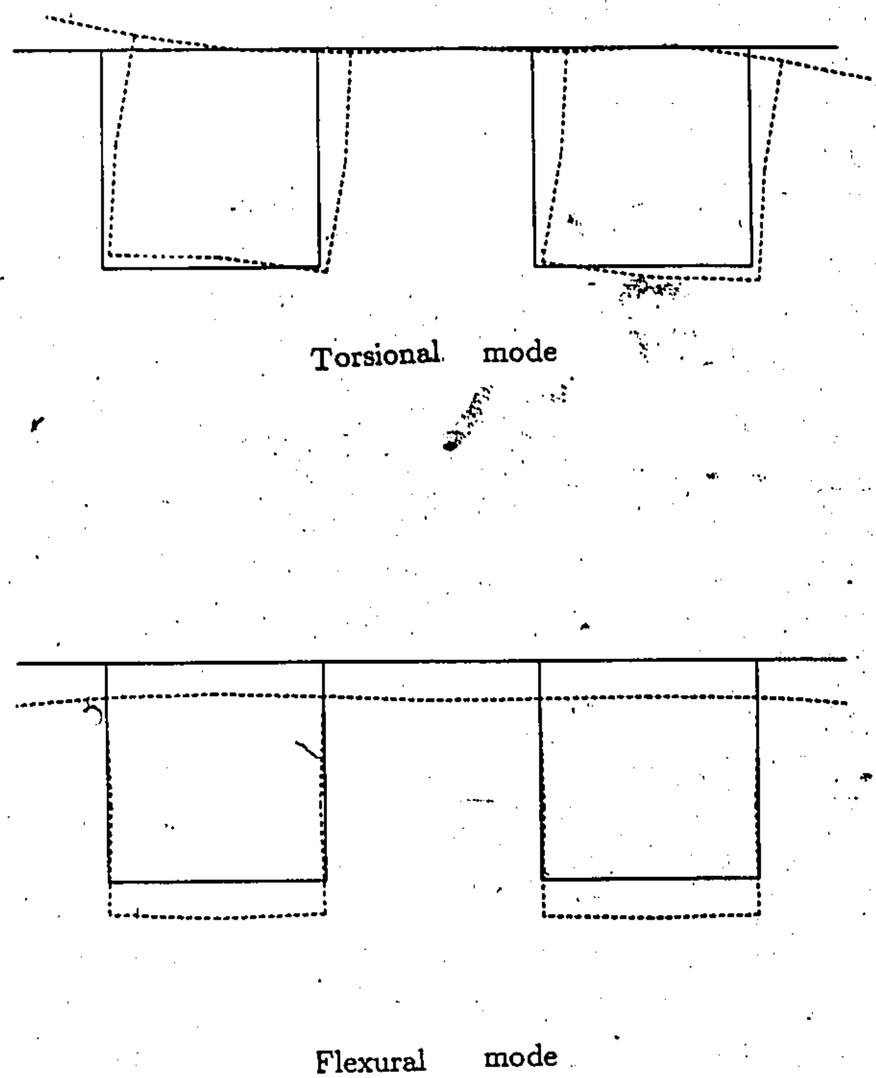


Figure 5.1: An Example of the Two Mode Types

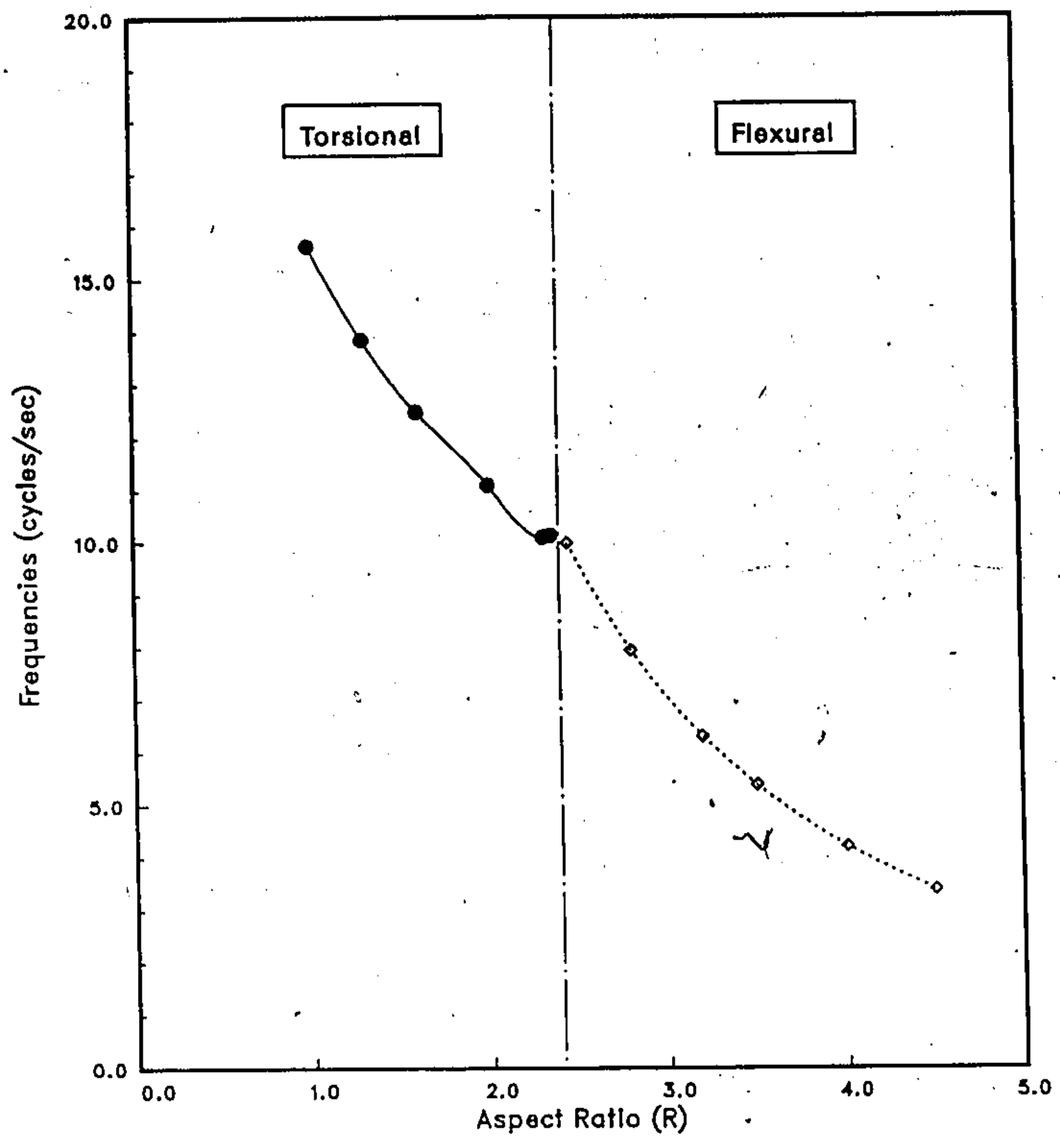


Figure 5.2: Effect of Aspect Ratio on First Natural Frequency (First Boundary)

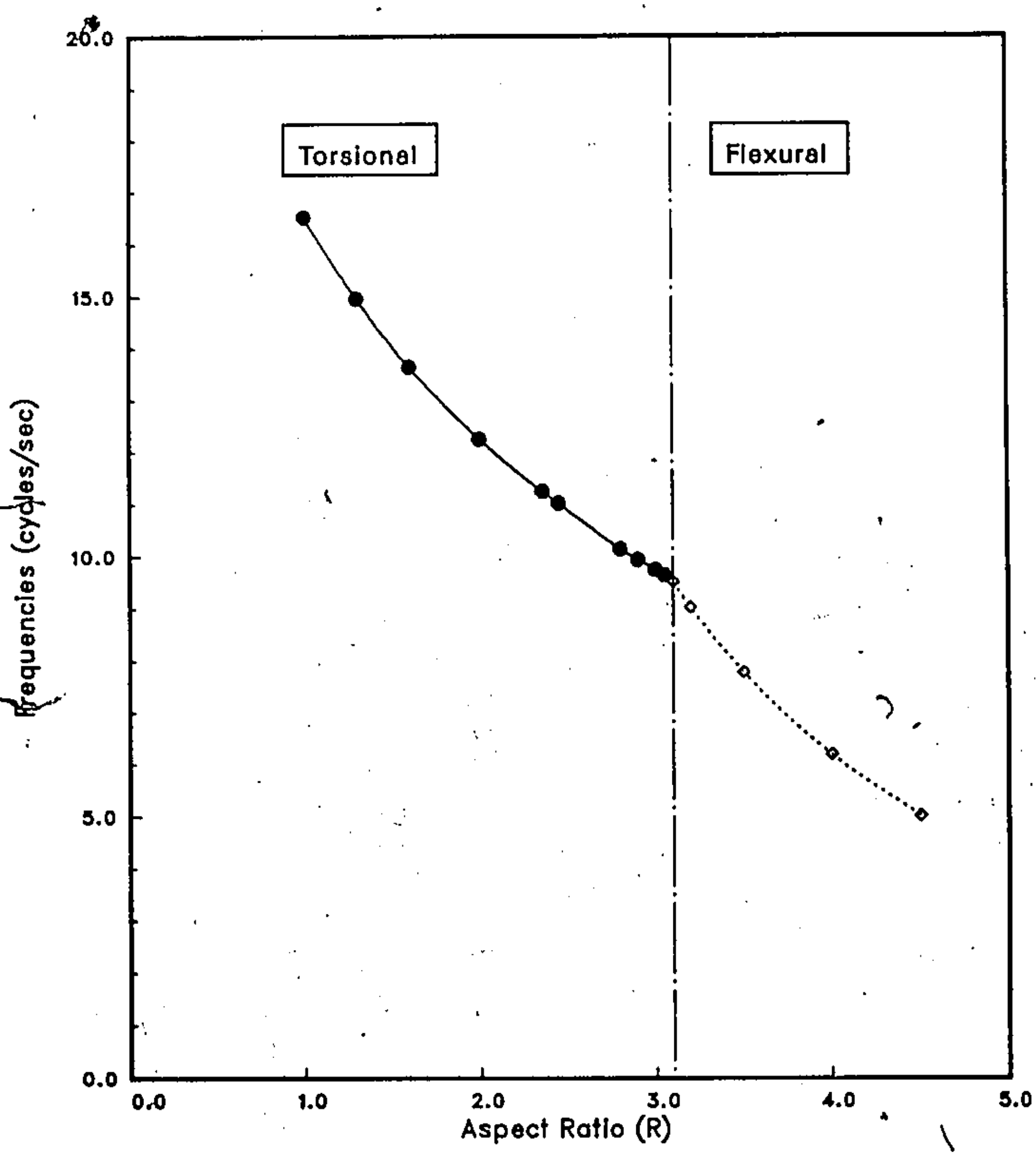


Figure 5.3: Effect of Aspect Ratio on First Natural Frequency (Second Boundary)

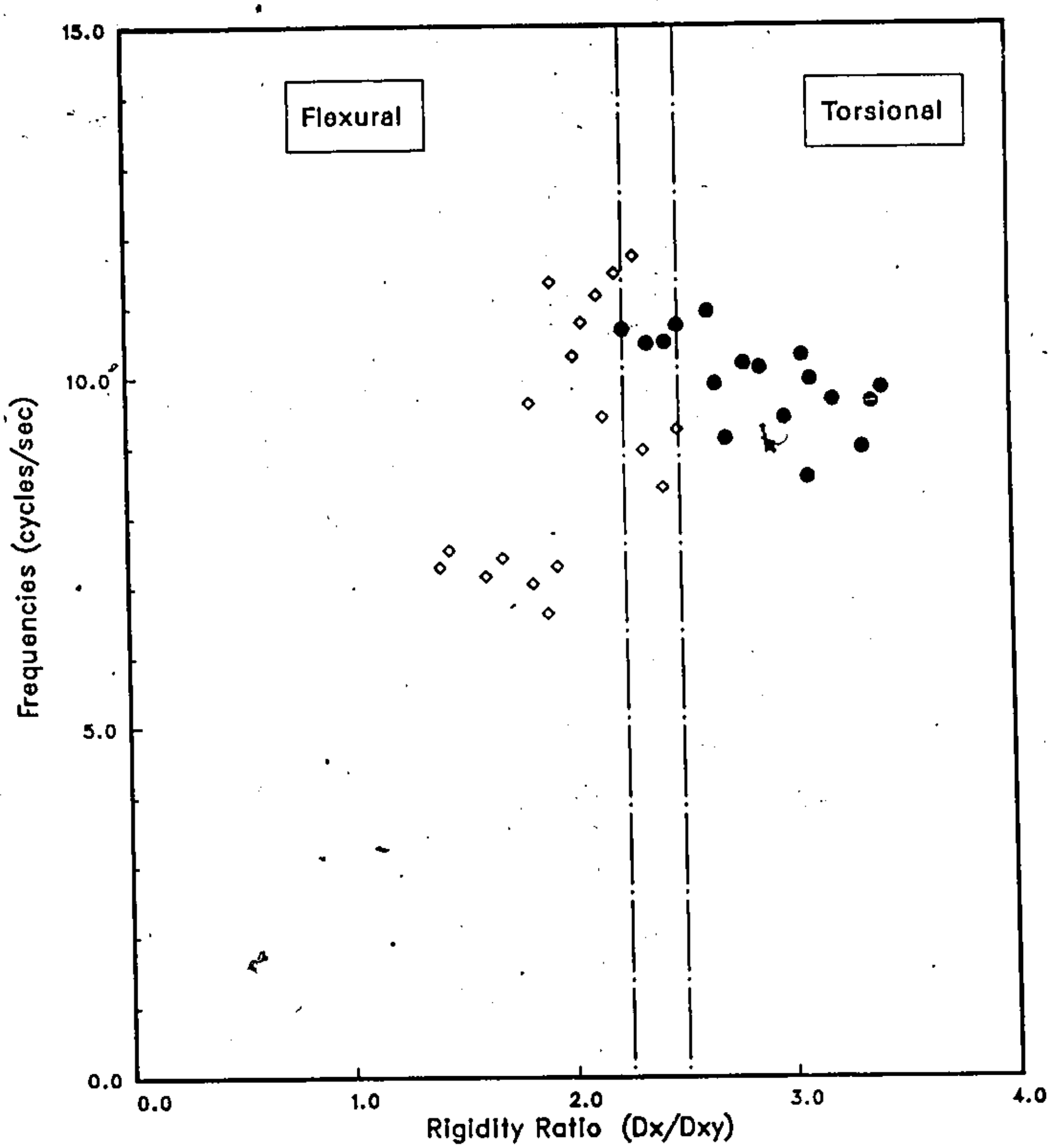


Figure 5.4: Effect of Rigidity Ratio on First Mode Type

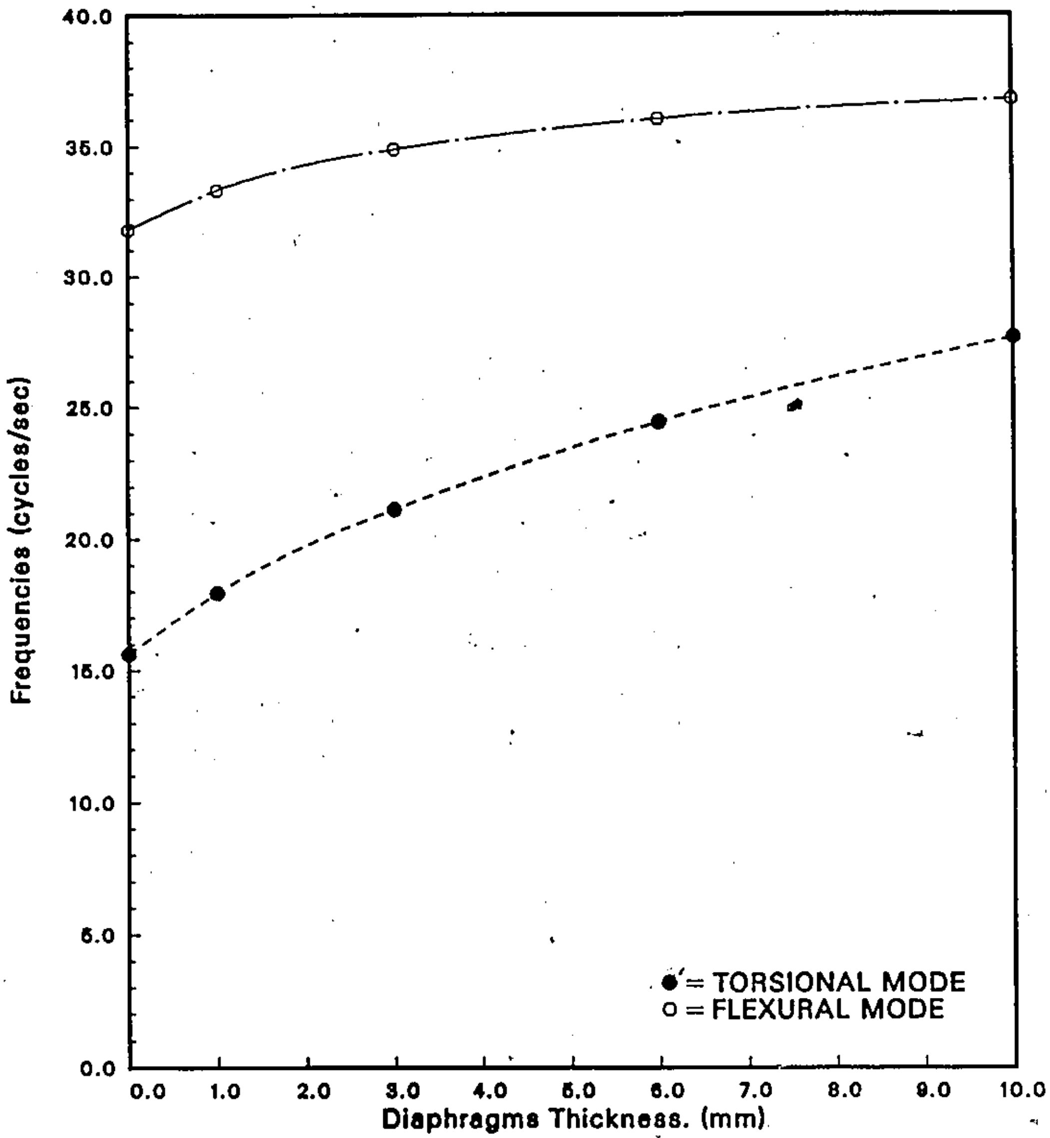


Figure 5.5: Effect of Diaphragms on Natural Frequency (Ratio R=1.00)

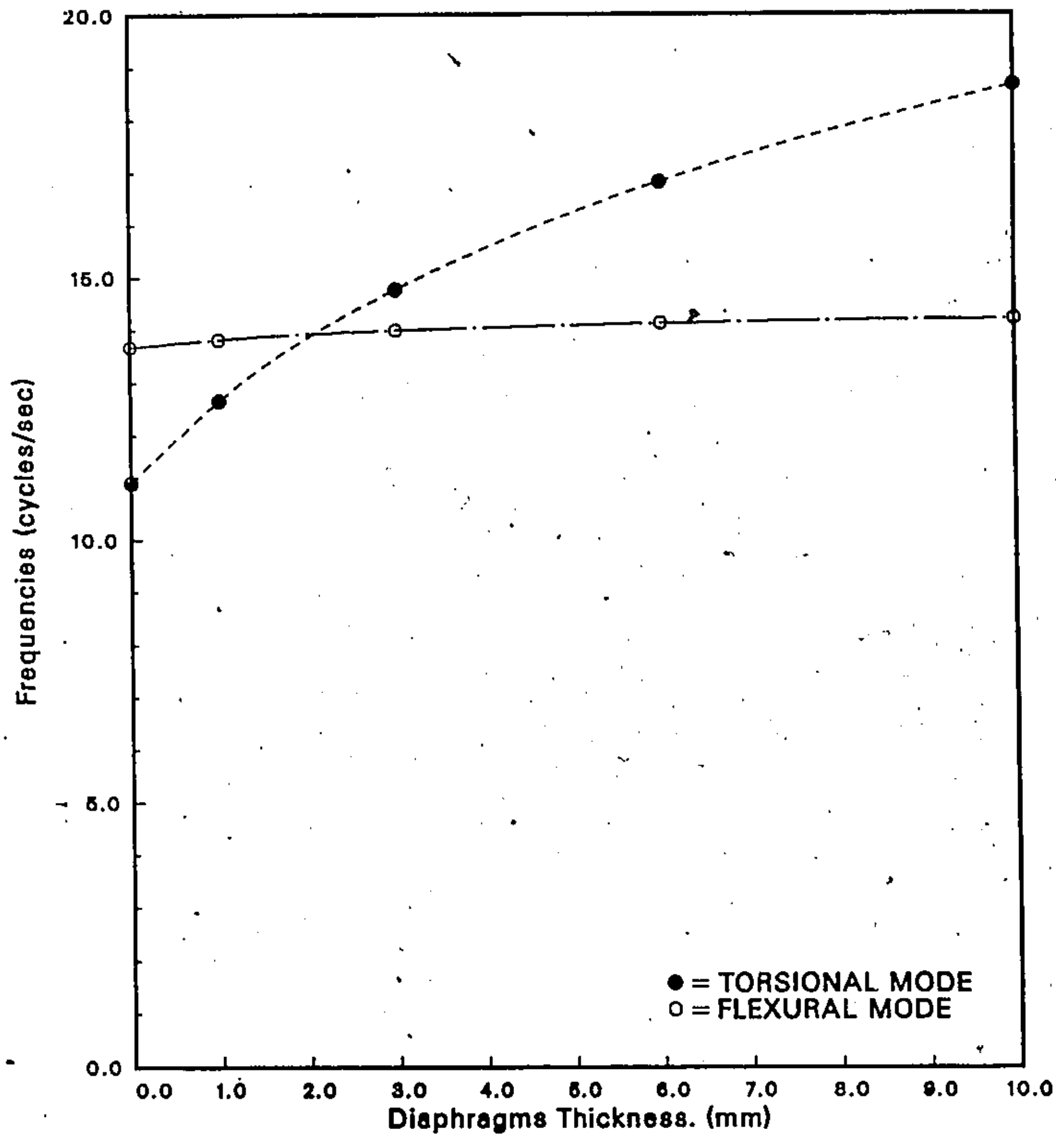


Figure 5.6: Effect of Diaphragms on Natural Frequency (Ratio R=2.00)

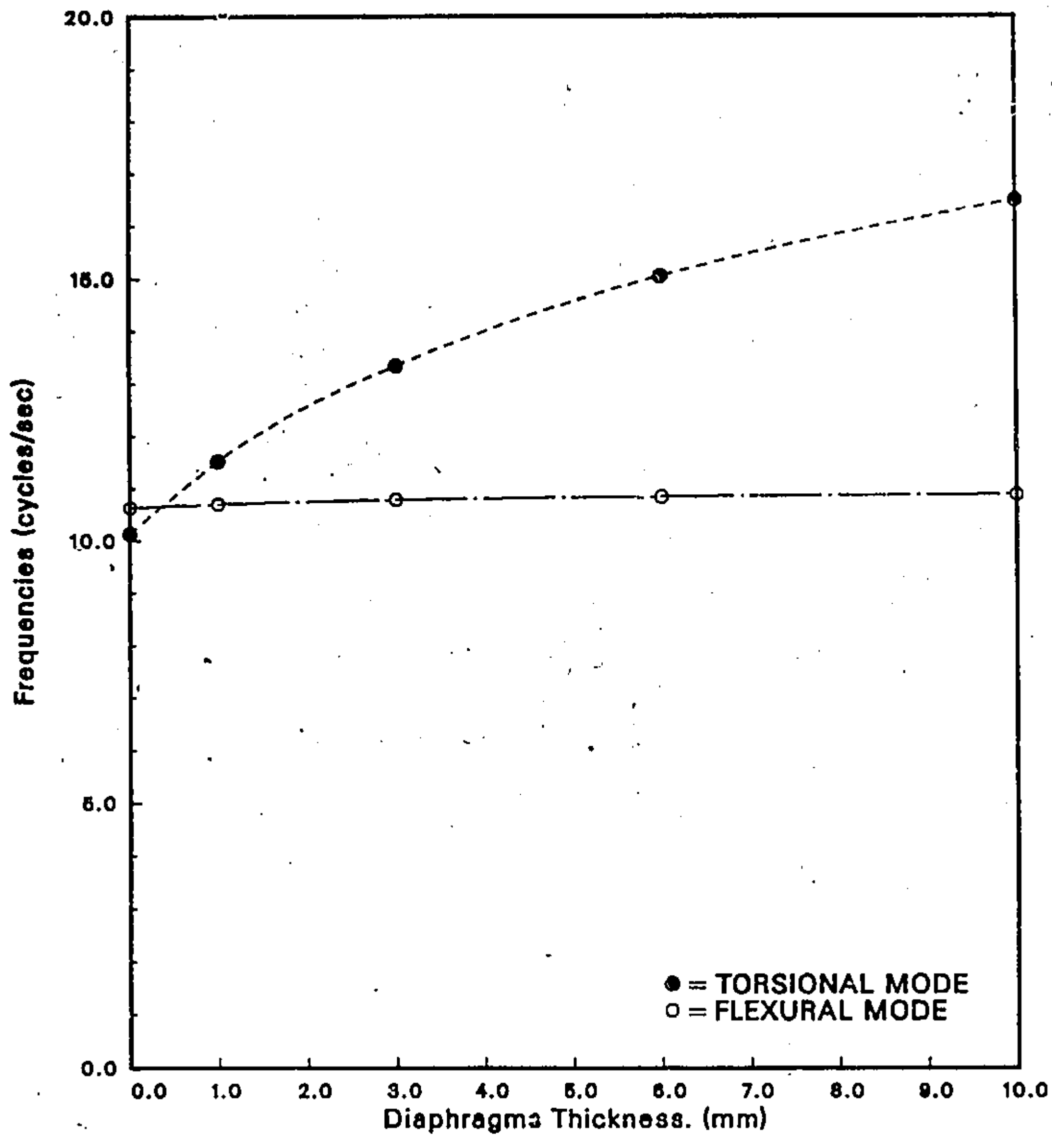


Figure 5.7: Effect of Diaphragms on Natural Frequency (Ratio R=2.353)

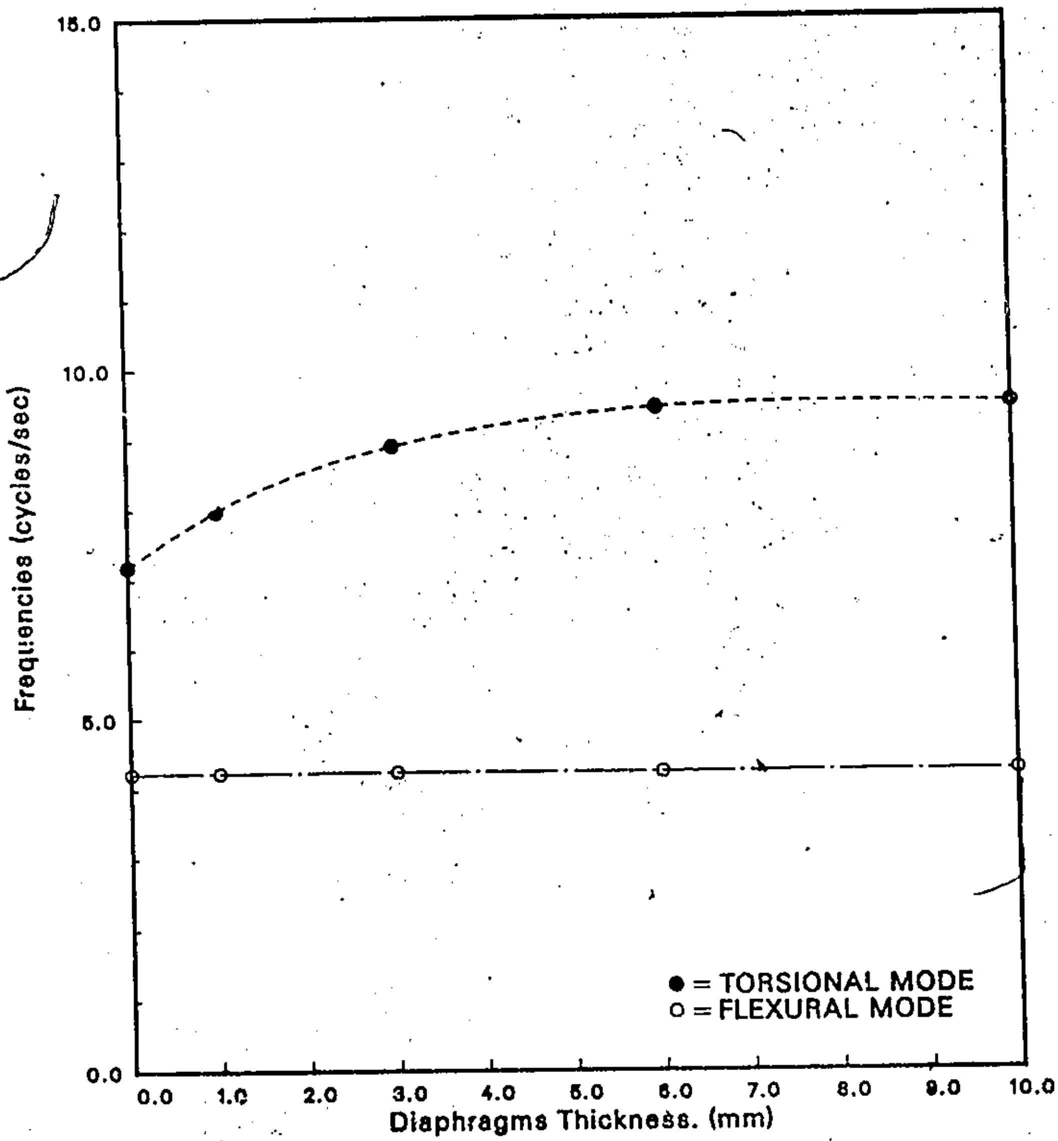


Figure 5.8: Effect of Diaphragrns on Natural Frequency (Ratio R=4.00)

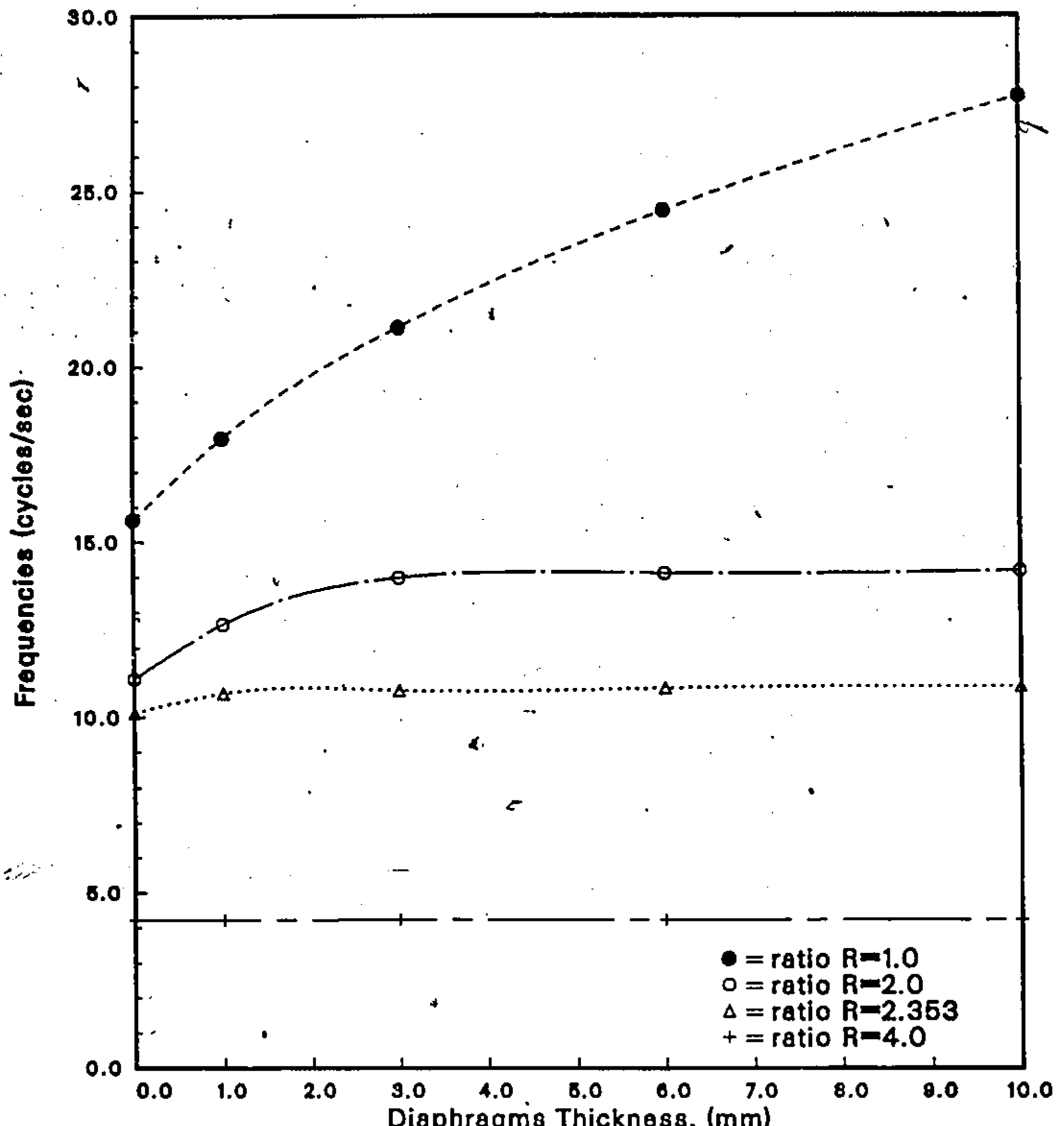


Figure 5.9: Effect of Diaphragms on First Natural Frequency (For Different Aspect Ratios)

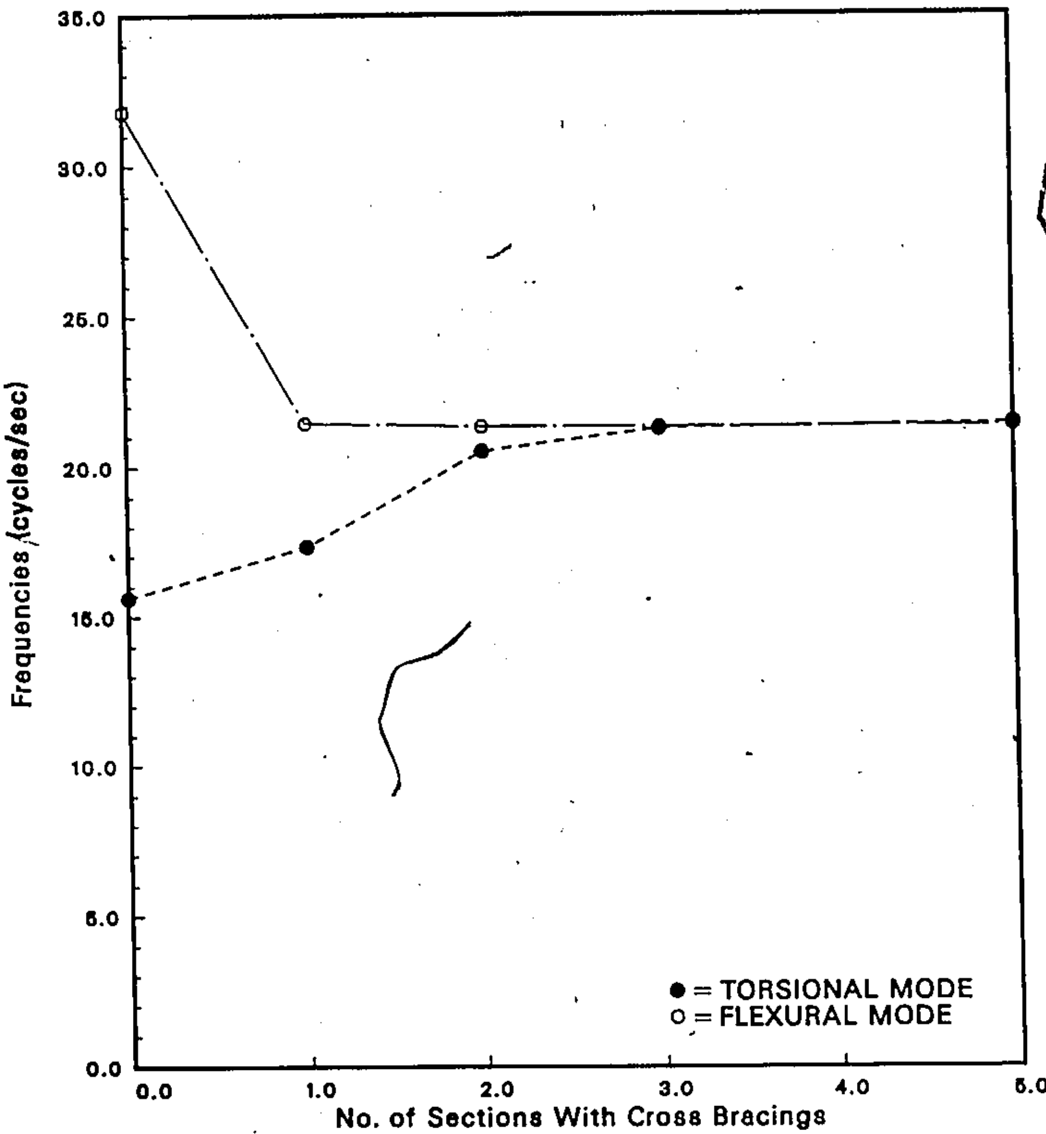


Figure 5.10: Effect of Bracings on Natural Frequency (Ratio R=1.00)

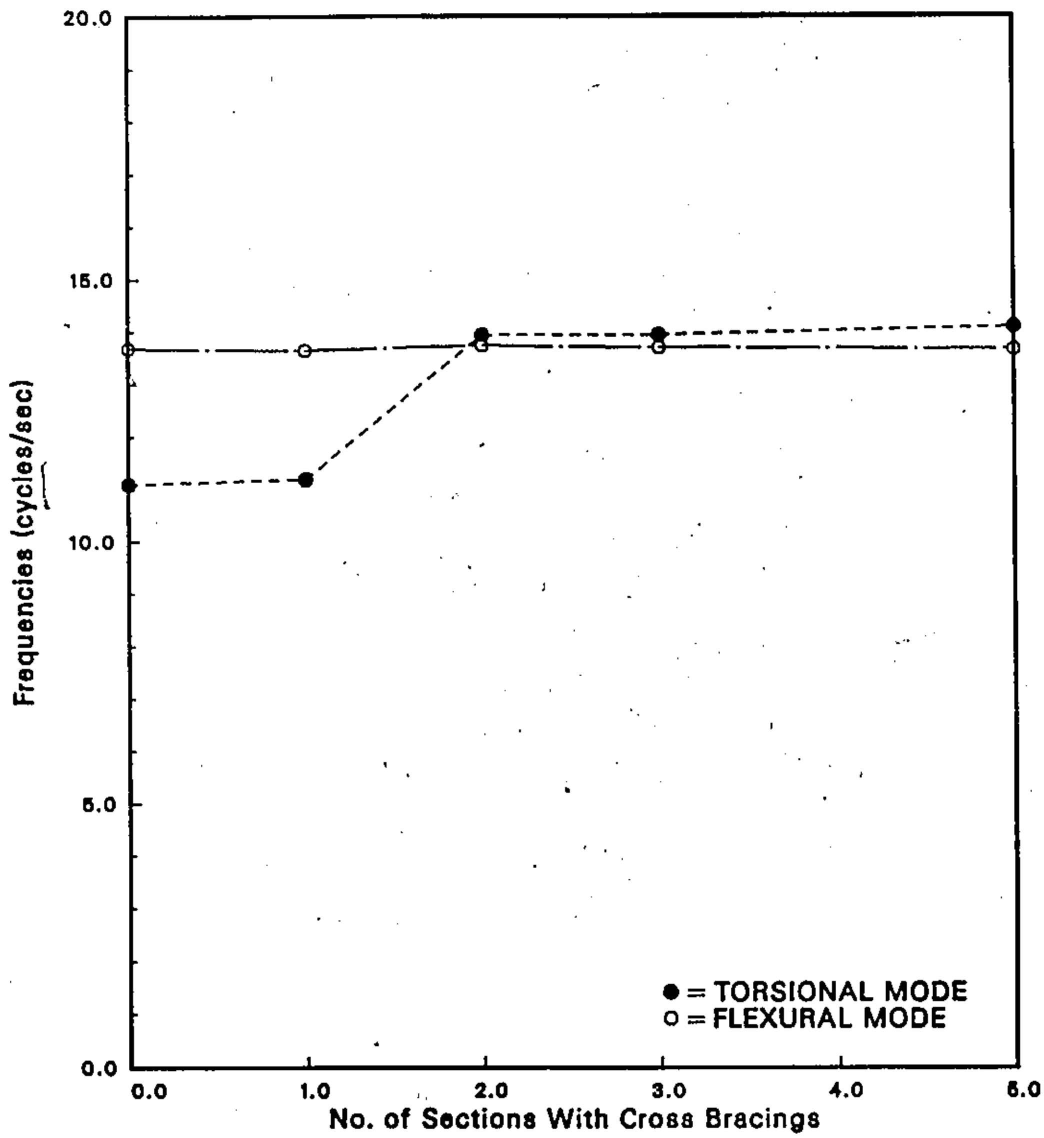


Figure 5.11: Effect of Bracings on Natural Frequency (Ratio R=2.00)

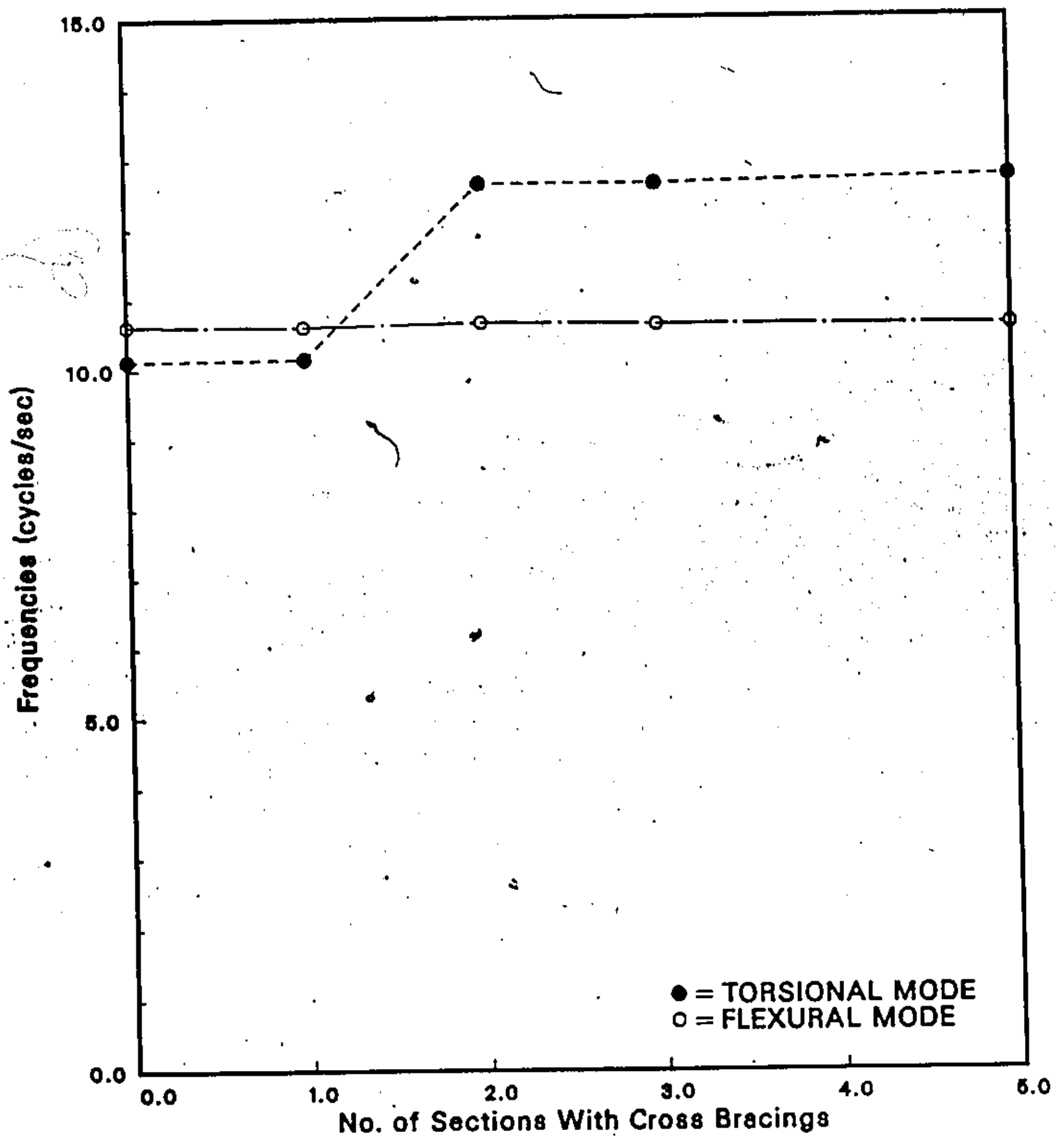


Figure 5.12: Effect of Bracings on Natural Frequency (Ratio R=2.353)

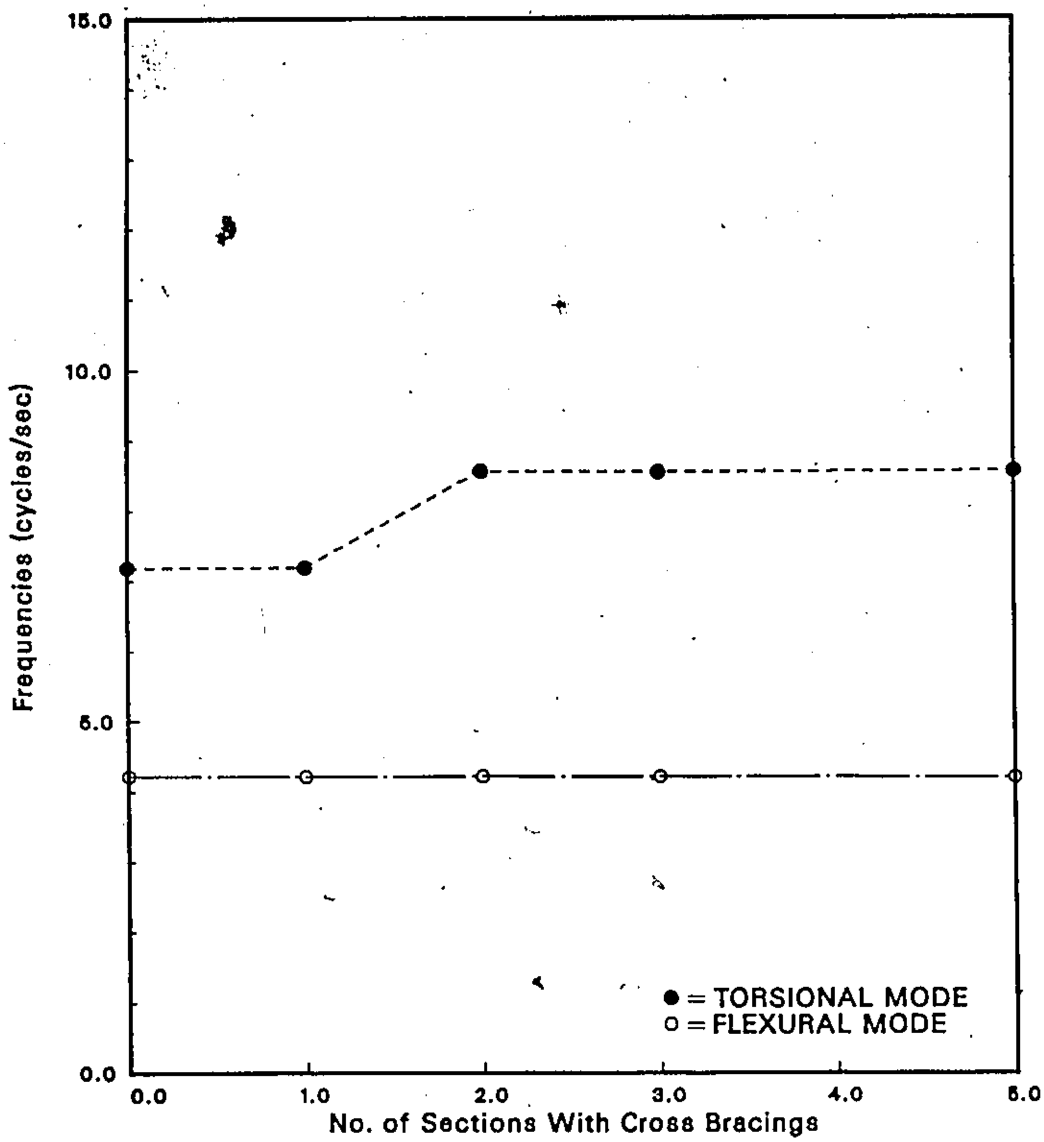


Figure 5.13: Effect of Bracings on Natural Frequency (Ratio R=4.00)

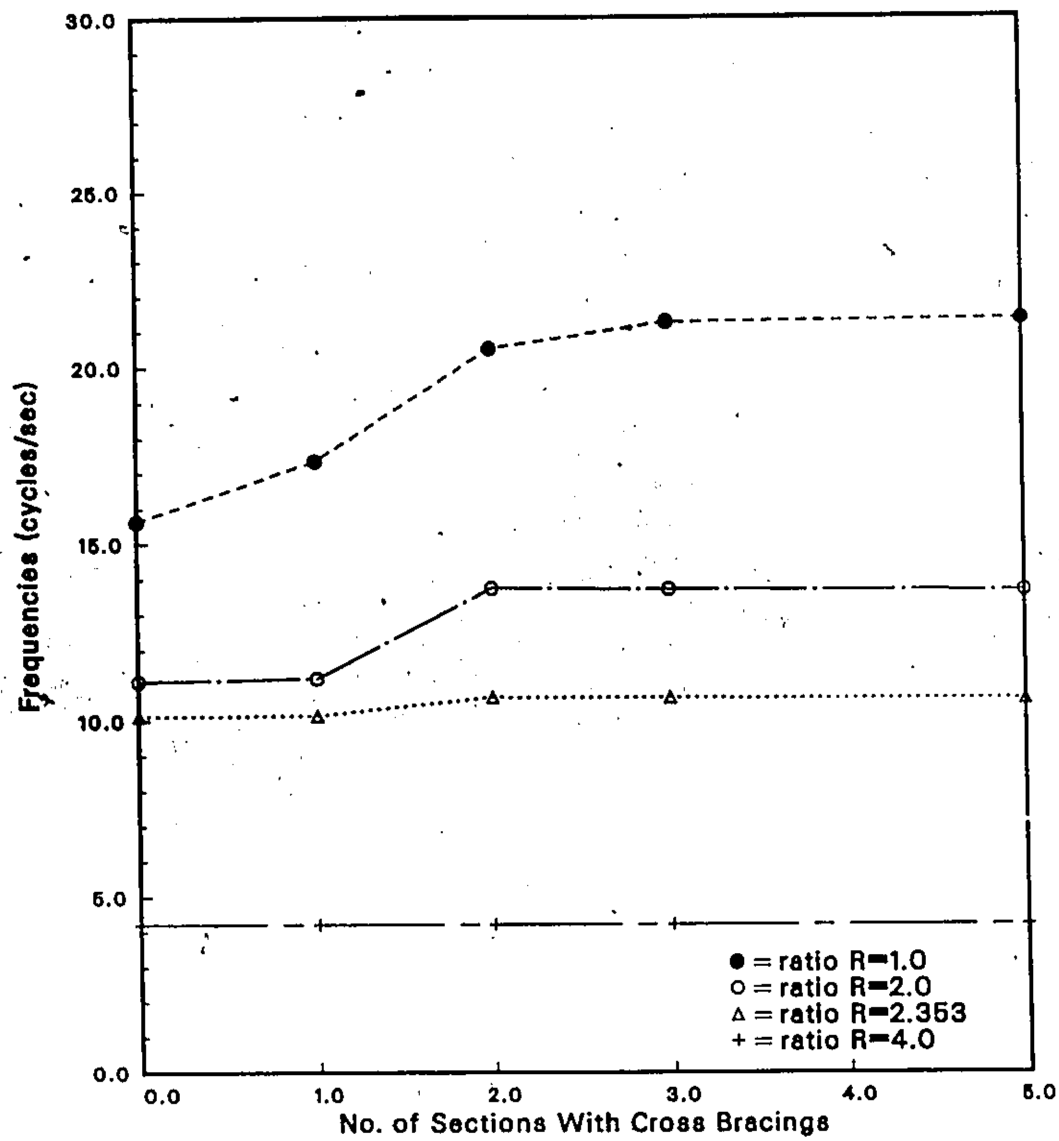


Figure 5.14: Effect of Bracings on First Natural Frequency (For Different Aspect Ratios)

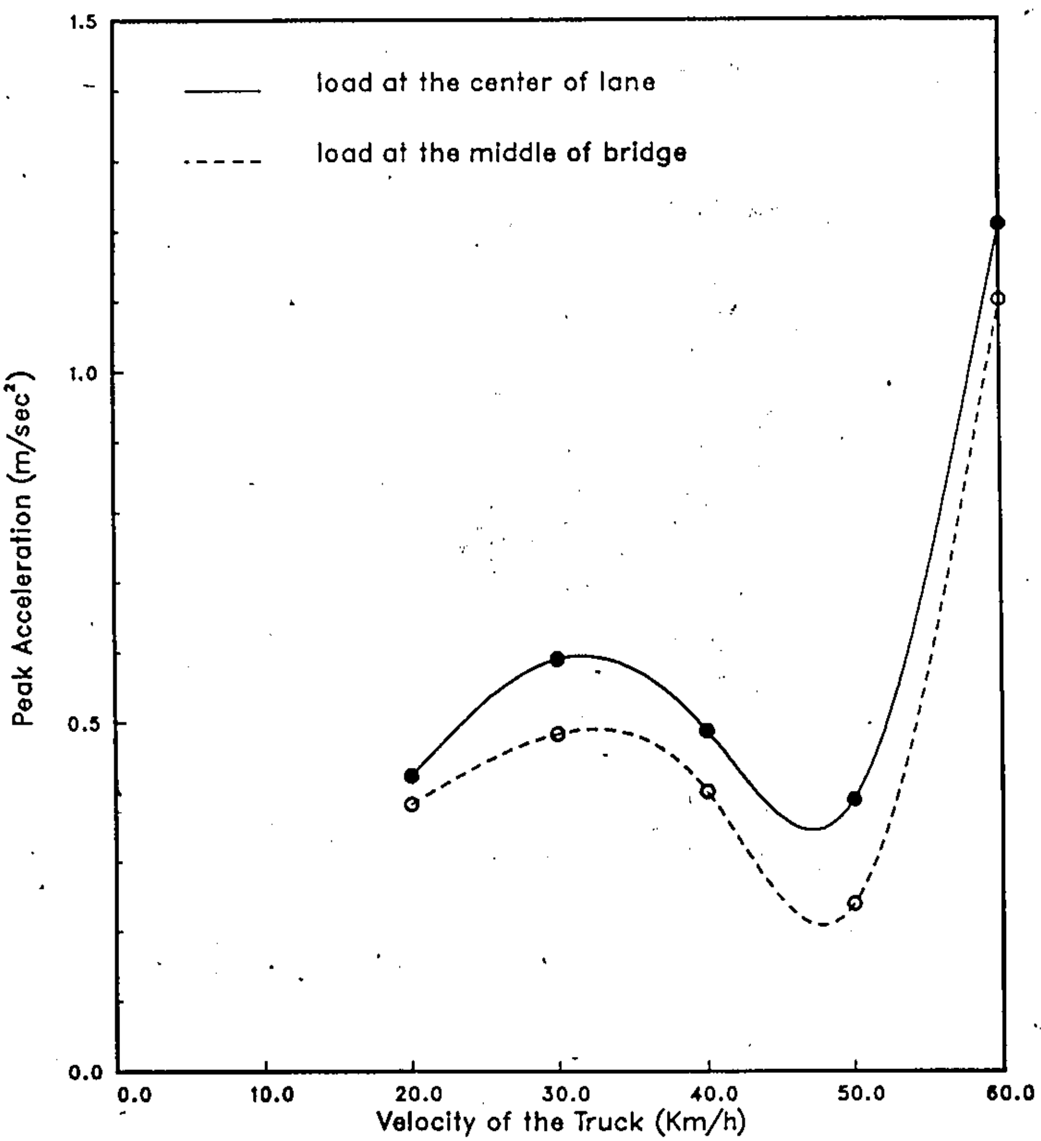


Figure 5.15: Effect of The Truck Speed on the Peak Acceleration

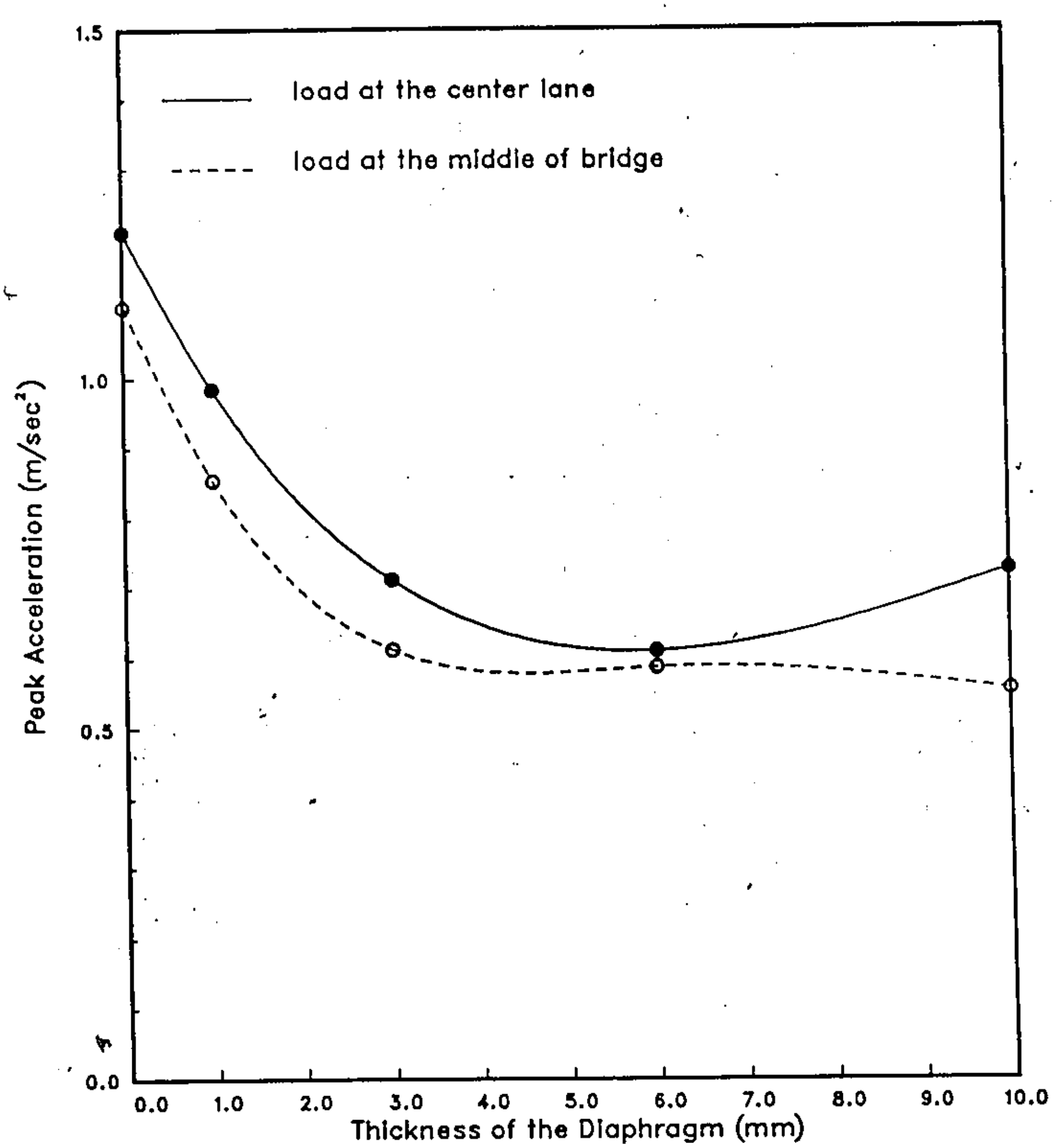


Figure 5.16: Effect of Diaphragms on the Peak Acceleration

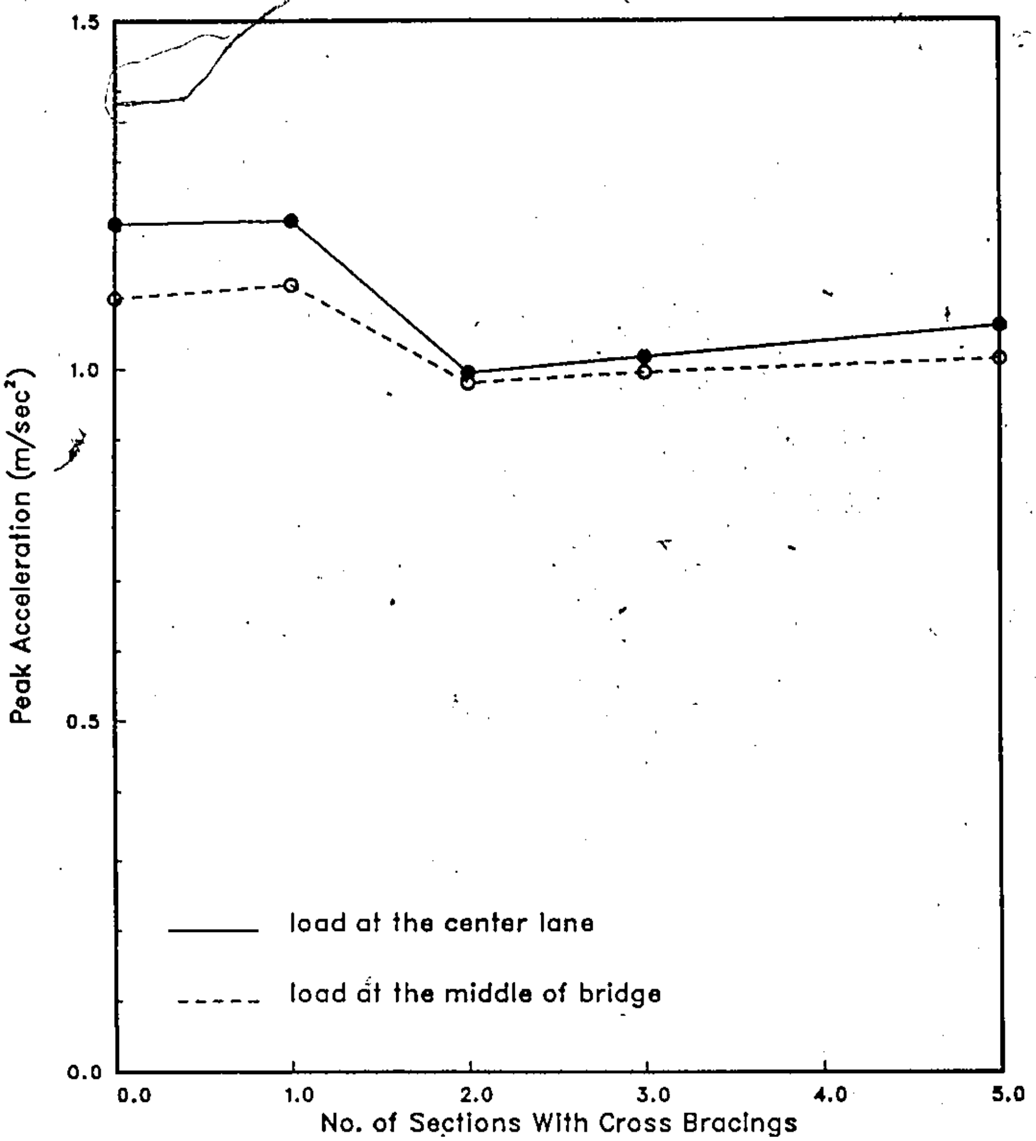


Figure 5.17: Effect of Bracings on the Peak Acceleration

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

The linear dynamic analysis of straight simply supported twin box girder bridge was successfully carried out by the finite element method. The commercially available finite element program (ADINA) was used to determine the free and forced vibration responses of the bridge.
It is observed that peak accelerations and dynamic deflections are usually more pronounced for torsional dominant modes of vibrations.

Since the dominant mode type and peak accelerations are closely related to the human response to vibration of bridges, the intend of the thesis was to concentrate the investigation on the parameters that affect the change in the dominant mode types (change from torsional to flexural) for free vibration and the change in the peak accelerations for the forced vibration

analysis.
Hence by avoiding torsional modes, the human unpleasantness will be reduced and the pedestrians comfort can be achieved.

Because the parameters investigated in this thesis were restricted to twin box girder bridges the conclusions cannot be generalized or interpreted for other types of bridges. Most of these data reported in this thesis is new to the field.
The intend of this study is to help design engineer to understand the basic response of a box girder bridge and to suggest possible ways in reducing acceleration levels rather than making design recommendations.

The conclusions drawn from the present study are strictly applicable to those bridge configurations analyzed. However, the finding of this study may be used someday in a further study to generalize dynamic response of box girder bridges and to set tables that can be used directly in determining the dominant mode type. Hence by simply calculating the values of a few parameters and using these tables we can avoid the torsional dominant mode or reduce the peak accelerations, factors that affect the human intolerance.

6.1 Conclusions

The main conclusions drawn directly from this study were as follows :

1. The linear dynamic finite element method is a powerful tool for the analysis of composite box girder bridges for either free or forced vibration responses.

2. In free vibration analysis, the change in the governing mode type is affected by several key parameters, such as aspect ratio and rigidity ratio.

   - For the bridge geometry and material property considered in this investigation, the first natural frequency decreases with increase in aspect ratios and the corresponding mode type changes from torsional to flexural at aspect ratios between 2.4 and 3.1 depending on the boundary conditions.

   - Using the curve fitting method, the frequency is related to the aspect ratio by the equation $f = 49.216R^{-1.7727}$ when the governing mode is torsional and by the equation $f = 15.78R^{-0.5177}$ when the governing mode is flexural for the first type of boundary conditions (f in cycles/sec).

   - For an aspect ratio around 2.4, the governing mode type changes from flexural to torsional type (or vice versa) depending on the rigidity ratio. Up to a rigidity ratio of 2.25 the governing mode type is flexural while for a rigidity ratio greater than 2.5 the governing mode type is torsional. The region where the rigidity ratio is less than 2.5 and greater than 2.25 the governing mode is either flexural or torsional.

3. Depending on the aspect ratio, the governing mode type was affected by the provision of diaphragms within the boxes at each support. The

first natural frequency was increased significantly due to the presence of diaphragms at the supports.

For an aspect ratio of 1.0 (where the first mode is torsional) the increase was 77 %, for a ratio of 2.353 (its first mode was torsional without diaphragms) the increase was 8.0 % and the governing mode changed to flexural mode, and for a ratio of 4.0 (where the first mode was flexural without diaphragms) the increase is less than 1.0 %. Then, the increase in first natural frequency is considerable when the governing mode is torsional.

4. The provision of bracing systems between boxes affects also the first mode type. Due to the presence of 5 cross bracings and for an aspect ratio of 1.0 the first natural frequency was increased by 29 % and the governing mode type was changed from torsional to flexural. For a ratio of 4.0 (where the governing mode is always flexural) there is a very small change (less than 0.2 %) in the first natural frequency due to the presence of bracing.

5. For the forced vibration study, the response is more important when the truck is in the middle of the lane than in the middle of the bridge.

6. The peak acceleration is changing harmonically with the speed of the truck for both cases. But the maximum value occurs at a speed of 60 Km/h.

7. The provision of diaphragms within the boxes at each support reduces considerably the peak acceleration for both cases. Due to 10 mm diaphragms the acceleration has been reduced by 40 % when the truck was in the center of the lane and 50 % when it was in the middle of

the bridge. Note that initially the governing mode was torsional. However for a case where the governing mode is flexural, the provision of diaphragms had an insignificant effect on the bridge response (variation of 2%).

8. The provision of bracing systems between the boxes can affect the peak acceleration only when they are placed at the supports. By adding a cross bracing at each support the peak acceleration decreases by 18.2 % when the truck was in the center of the lane and by 12.6 % when it was in the middle of the bridge. Any bracing placed between the supports has a very small effect on the peak acceleration (variation of 0.36 %). Initially the governing mode was torsional.

6.2 Recommendations for Future Work

In this thesis the analysis was restricted to straight simply supported twin box girder bridges. The conclusions drawn from this study are specific for that case and further studies could be directed towards that of obtaining general solutions for free or forced vibration problems of other box girder bridges rather than straight one, such as :

- Curved bridges.
- Skew bridges.

The effect of aspect ratio could be generalized for other values of rigidity ratios.
Other parameters affecting the change in the first mode type can be investigated to complete the study such as spacing of the box girders or the increase in the number of the boxes ...
The same analysis can be used for continuous box girder bridges where other parameters may influence the first mode type. e.g Spanlength ratio (Ratio of lengths of a two span continuous bridge).

For the forced vibration study further analytical work could be undertaken to consider the effect of other parameters such as :

- Spacing of the boxes.
- Increase in the number of the boxes.
- change in aspect ratio
- change in rigidity ratio.

Finally additional experimental studies are required to better understand the linear dynamic response of box girder bridges because the confidence of any analytical model can only be established by comparison with the experimental data generated from model studies and from the field tests.

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Appendix A

Sample Input for ADINA Analysis

```
*****************************************************************
*                                                               *
*  FIRST EXAMPLE: FREE VIBRATION OF A SIMPLY SUPPORTED BEAM     *
*                 USING THE ISOPARAMETRIC BEAM ELEMENT.         *
*                                                               *
*****************************************************************

'PLOTSIZE' 30.0 27.5
DATABASE CREATE
HEADING 'FREE VIBRATION OF A SIMPLY SUPPORTED BEAM BEAM ELEMENT'
MASTER IDOF=101110
ANALYSIS TYPE=DYNAMIC MASS=CONSISTENT IMODS=1 NMODES=4
FREQUENCIES SUB NEIG=4 NMODE=4
PRINTOUT MINIMUM
PRINTNODE 1 22 1
COORDINATES
ENTRIES NODE X Y
1 0. 0.
TO
21 45.72 0.
22 0. 1.
MATERIAL 1 ELASTIC E=2.E11 NU=0.3 DEN=7800.
*
* BEAM ELEMENTS
*
EGROUP 1 BEAM M=1
SECTION 1 PROP RI=1. SI=19.9824 TI=.828113 A=1.00513 SA=0 TA=0
ENODES
1 22 1 2
TO
20 22 20 21
LIST ENODES 1 20
EDATA
ENTRIES EL SECTION
1 1
TO
20 1
BOUNDARIES IDOF=111111
22
BOUNDARIES IDOF=111110
1 21
FRAME
MESH
ADINA OPTIM=YES
LIST COORD 1 22
END
```

```
*****************************************************************
*                                                               *
* SECOND EXAMPLE : FREE VIBRATION OF A TWO BOX BRIDGE (RATIO=1.0)  *
*                  WITHOUT BRACING OR DIAPHRAGMS.               *
*                  441 NODES AND 110 ELEMENTS.                  *
*                                                               *
*****************************************************************
'PLOTSIZE' 30.0 27.5
DATABASE CREATE
HEADING 'SIMPLE 2-BOX WITHOUT BRACING LENGHT=9.35M BOX H=2.5 '
MASTER IDOF=000000
ANALYSIS TYPE=DYNAMIC MASS=CONSISTENT
FREQUENCIES SUB NEIG=2 NMODE=2 SSTOL=1.0-3
PRINTOUT MINIMUM
PRINTNODES 1 441 1
COORDINATES
ENTRIES NODE X Y Z
*****************
* CONCRETE DECK *
*****************
1 -4.675 9.35 2.5
21 -4.675 0. 2.5
43 -3.675 9.35 2.5
63 -3.675 0. 2.5
169 3.675 9.35 2.5
189 3.675 0. 2.5
211 4.675 9.35 2.5
231 4.675 0. 2.5
SHELLNODES DIRECTOR=-2 SYSTEM-D=L
43 -2 G TO 63 -2 G
85 -2 G TO 105 -2 G
127 -2 G TO 147 -2 G
169 -2 G TO 189 -2 G
232 -2 G TO 252 -2 G
274 -2 G TO 294 -2 G
295 -2 G TO 315 -2 G
337 -2 G TO 357 -2 G
LINE STRAIGHT 1 21 EL=10 MID=1 NF=2
LINE STRAIGHT 1 43 EL=1 MID=1 NF=22 NSTEP=21
LINE STRAIGHT 43 169 EL=3 MID=1 NF=64 NSTEP=21
LINE STRAIGHT 169 211 EL=1 MID=1 NF=190 NSTEP=21
LINE STRAIGHT 211 231 EL=10 MID=1 NF=212
LINE STRAIGHT 21 63 EL=1 MID=1 NF=42 NSTEP=21
LINE STRAIGHT 63 189 EL=3 MID=1 NF=84 NSTEP=21
LINE STRAIGHT 189 231 EL=1 MID=1 NF=210 NSTEP=21
*
LINE COMBINE 1 211 43 169
LINE COMBINE 21 231 63 189
*
MATERIAL 1 ELASTIC E=265.E8 NU=0.15 DEN=2400. K=.8333
MATERIAL 2 ELASTIC E=2.E11 NU=0.3 DEN=7800. K=.8333
*
EGROUP 1 SHELL M=1 RINT=3 SINT=3 TINT=2
```

```
GSURFACE 1 21 231 211 EL1=10 EL2=5 NODES=9 NF=2 EF=1
THICKNESS 1 0.27
STRESSTABLE 1 1 2 3 4
EDATA
ENTRIES EL NTH TABLE
1 1 1
TO
50 1 1
LIST ENODES 1 50
***************
* STEEL BOXES *
***************
EGROUP 2 SHELL M=2 RINT=3 SINT=3 TINT=2
COORDINATES
ENTRIES NODE X Y Z
232 -3.675 9.35 0.
253 -2.450 9.35 0.
274 -1.225 9.35 0.
295 1.225 9.35 0.
316 2.450 9.35 0.
337 3.675 9.35 0.
358 -3.675 9.35 1.25
379 -1.225 9.35 1.25
400 1.225 9.35 1.25
421 3.675 9.35 1.25
NGENERATION TIMES=20 NSTEP=1 YSTEP=-0.4675
232 253 274 295 316 337 358 379 400 421
ENODES
ENTRIES EL N1 N2 N3 N4 N5 N6 N7 N8 N13
1 232 234 276 274 233 255 275 253 254
11 295 297 339 337 296 318 338 316 317
21 43 45 234 232 44 360 233 358 359
31 85 87 276 274 86 381 275 379 380
41 127 129 297 295 128 402 296 400 401
51 169 171 339 337 170 423 338 421 422
EGENERATION TIMES=9 ESTEP=1 NSTEP1=2
1 11 21 31 41 51
THICKNESS 2 0.012
THICKNESS 3 0.025
STRESSTABLE 1 1 2 3 4
EDATA
ENTRIES EL NTH TABLE
21 2 1
TO
60 2 1
1 3 1
TO
20 3 1
LIST ENODES 1 60
BOUNDARIES IDOF=111000
252 273 294 315 336 357
BOUNDARIES IDOF=101000
232 253 274 295 316 337
```

```
FRAME
MESH
ADINA OPTIM=YES
LIST COORD 1 441
END
```

```
**********************************************************************
*                                                                    *
*  THIRD EXAMPLE : FREE VIBRATION OF A SIMPLY SUPPORTED BRIDGE       *
*                  WITH 5 CROSS BRACINGS.                            *
*                  451 NODES, 110 SHELL ELEMENTS AND 25 BEAM ELEMENTS *
*                                                                    *
**********************************************************************
'PLOTSIZE' 30.0 27.5
DATABASE CREATE
HEADING 'SIMPLE 2-BOX GIRDER WITH 5 CROSS BRAC LENGHT=18.7M '
MASTER IDOF=000000
ANALYSIS TYPE=DYNAMIC MASS=CONSISTENT
FREQUENCIES SUB NEIG=2 NMODE=2 SSTOL=1.0E-4
PRINTOUT MINIMUM
PRINTNODES 1 451 1
COORDINATES
ENTRIES NODE X Y Z
*****************
* CONCRETE DECK *
*****************
1 -4.675 18.7 2.5
21 -4.675 0. 2.5
43 -3.675 18.7 2.5
63 -3.675 0. 2.5
169 3.675 18.7 2.5
189 3.675 0. 2.5
211 4.675 18.7 2.5
231 4.675 0. 2.5
SHELLNODES DIRECTOR=-2 SYSTEM-D=L
43 -2 G TO 63 -2 G
85 -2 G TO 105 -2 G
127 -2 G TO 147 -2 G
169 -2 G TO 189 -2 G
232 -2 G TO 252 -2 G
274 -2 G TO 294 -2 G
295 -2 G TO 315 -2 G
337 -2 G TO 357 -2 G
LINE STRAIGHT 1 21 EL=10 MID=1 NF=2
LINE STRAIGHT 1 43 EL=1 MID=1 NF=22 NSTEP=21
LINE STRAIGHT 43 169 EL=3 MID=1 NF=64 NSTEP=21
LINE STRAIGHT 169 211 EL=1 MID=1 NF=190 NSTEP=21
LINE STRAIGHT 211 231 EL=10 MID=1 NF=212
LINE STRAIGHT 21 63 EL=1 MID=1 NF=42 NSTEP=21
LINE STRAIGHT 63 189 EL=3 MID=1 NF=84 NSTEP=21
LINE STRAIGHT 189 231 EL=1 MID=1 NF=210 NSTEP=21
*
LINE COMBINE 1 211 43 169
LINE COMBINE 21 231 63 189
*
MATERIAL 1 ELASTIC E=265.E8 NU=0.15 DEN=2400. K=.8333
MATERIAL 2 ELASTIC E=2.E11 NU=0.3 DEN=7800. K=.8333
EGROUP 1 SHELL M=1 RINT=3 SINT=3 TINT=2
GSURFACE 1 21 231 211 EL1=10 EL2=5 NODES=9 NF=2 EF=1
```

```
THICKNESS 1 0.27
STRESSTABLE 1 1 2 3 4
EDATA
ENTRIES EL NTH TABLE
1 1 1
TO
50 1 1
LIST ENODES 1 50
****************
* STEEL BOXES *
****************
EGROUP 2 SHELL M=2 RINT=3 SINT=3 TINT=2
COORDINATES
ENTRIES NODE X Y Z
232 -3.675 18.7 0.
253 -2.450 18.7 0.
274 -1.225 18.7 0.
295 1.225 18.7 0.
316 2.450 18.7 0.
337 3.675 18.7 0.
358 -3.675 18.7 1.25
379 -1.225 18.7 1.25
400 1.225 18.7 1.25
421 3.675 18.7 1.25
NGENERATION TIMES=20 NSTEP=1 YSTEP=-0.935
232 253 274 295 316 337 358 379 400 421
ENODES
ENTRIES EL N1 N2 N3 N4 N5 N6 N7 N8 N13
1 232 234 276 274 233 255 275 253 254
11 295 297 339 337 296 318 338 316 317
21 43 45 234 232 44 360 233 358 359
31 85 87 276 274 86 381 275 379 380
41 127 129 297 295 128 402 296 400 401
51 169 171 339 337 170 423 338 421 422
EGENERATION TIMES=9 ESTEP=1 NSTEP1=2
1 11 21 31 41 51
THICKNESS 2 0.012
THICKNESS 3 0.025
STRESSTABLE 1 1 2 3 4
EDATA
ENTRIES EL NTH TABLE
21 2 1
TO
60 2 1
1 3 1
TO
20 3 1
LIST ENODES 1 60
*****************
* CROSS BRACING *
*****************
EGROUP 3 BEAM M=2
COORDINATES
```

```
ENTRIES NODE X Y Z
442 0. 0. 1.25
443 0. 18.7 1.25
444 0. 9.35 1.25
445 0. 0. 5.0
446 0. 18.7 5.0
447 0. 9.35 5.0
448 0. 14.025 1.25
449 0. 4.675 1.25
450 0. 14.025 5.0
451 0. 4.675 5.0
*****************
* BEAM ELEMENTS *
*****************
SECTION 1 PROP RI=2.8625E-6 SI=1.0416E-6 TI=4.166E-6 AREA=5E-3,
SA=4.1667E-3 TA=4.1667E-3
ENODES
1 445 294 442
2 445 442 105
3 445 442 147
4 445 315 442
5 445 294 315
6 446 274 443
7 446 443 85
8 446 443 127
9 446 295 443
10 446 274 295
11 447 284 444
12 447 444 95
13 447 444 137
14 447 305 444
15 447 284 305
16 451 289 449
17 451 449 100
18 451 449 142
19 451 310 449
20 451 289 310
21 450 279 448
22 450 448 90
23 450 448 132
24 450 300 448
25 450 279 300
LIST ENODES 1 25
EDATA
ENTRIES EL SECTION
1 1
TO
25 1
BOUNDARIES IDOF=111111
445 446 447 450 451
BOUNDARIES IDOF=111000
252 273 294 315 336 357
BOUNDARIES IDOF=101000
```

```
232 253 274 295 316 337
FRAME
MESH
ADINA OPTIM=YES
LIST COORD 1 451
END
```

```
***********************************************************************
*                                                                     *
* FORTH EXAMPLE : FORCED VIBRATION OF A SIMPLY SUPPORTED BOX BRIDGE   *
*                 WITH A 10 MM DIAPHRAGM. LOAD AT CENTRE LANE.        *
*                 VELOCITY OF THE TRUCK = 60 KM/H.                    *
*                 781 NODES AND 202 SHELL ELEMENTS.                   *
*                 NO. OF STEPS=400 AND TIME INCREMENT=0.01 SEC.       *
*                                                                     *
***********************************************************************
'PLOTSIZE' 30.0 27.5
DATABASE CREATE
HEADING '2-BOX LENGHT=21.6H H=2.5 DIAPTH=10MM CENTRE LANE LOAD V60'
MASTER IDOF=000000 NSTEP=400 DT=.01
ANALYSIS TYPE=DYNAMIC MASS=CONSISTENT IM=1 NM=2
FREQUENCIES SUB NEIG=2 NMODE=0 SSTOL=1.0E-3
PRINTOUT VO=MAX CA=NO IPD=4 IOUT=1 IPRIC=0 IPRIT=0 IDC=1 IVC=0 IAC=1,
PR=NO
PRINTNODES 10 28 9 47 65 9 84 102 9 121 139 9 158 176 9 195 213 9,
232 250 9 269 287 9 306 324 9 343 361 9 380 398 9
PRINTSTEPS 10 400 10
PORTHOLE V=MAXIMUM N=0 JDC=0 JVC=0 JAC=1 JTC=0 S=YES
NSAVESTEPS 30 400 30
ESAVESTEPS 30 400 30
TIMEFUNCTION 1
0. 0.
0.072 -1.
0.144 0.
100. 0.
COORDINATES
ENTRIES NODE X Y Z
*****************
* CONCRETE DECK *
*****************
1 -4.675 21.6 2.5
37 -4.675 0. 2.5
75 -3.675 21.6 2.5
111 -3.675 0. 2.5
297 3.675 21.6 2.5
333 3.675 0. 2.5
371 4.675 21.6 2.5
407 4.675 0. 2.5
SHELLNODES DIRECTOR=-2 SYSTEM-D=L
75 -2 G TO 111 -2 G
149 -2 G TO 185 -2 G
223 -2 G TO 259 -2 G
297 -2 G TO 333 -2 G
408 -2 G TO 444 -2 G
482 -2 G TO 518 -2 G
519 -2 G TO 555 -2 G
593 -2 G TO 629 -2 G
112 -2 G
630 -2 G
445 -2 G
```

```
667 -2 G
260 -2 G
704 -2 G
556 -2 G
741 -2 G
148 -2 G
666 -2 G
481 -2 G
703 -2 G
296 -2 G
740 -2 G
592 -2 G
777 -2 G
LINE STRAIGHT 1 37 EL=18 MID=1 NF=2
LINE STRAIGHT 1 75 EL=1 MID=1 NF=38 NSTEP=37
LINE STRAIGHT 75 297 EL=3 MID=1 NF=112 NSTEP=37
LINE STRAIGHT 297 371 EL=1 MID=1 NF=334 NSTEP=37
LINE STRAIGHT 371 407 EL=18 MID=1 NF=372
LINE STRAIGHT 37 111 EL=1 MID=1 NF=74 NSTEP=37
LINE STRAIGHT 111 333 EL=3 MID=1 NF=148 NSTEP=37
LINE STRAIGHT 333 407 EL=1 MID=1 NF=370 NSTEP=37
*
LINE COMBINE 1 371 75 297
LINE COMBINE 37 407 111 333
*
MATERIAL 1 ELASTIC E=265.E8 NU=0.15 DEN=2400. K=.8333
MATERIAL 2 ELASTIC E=2.E11 NU=0.3 DEN=7800. K=.8333
EGROUP 1 SHELL M=1 RINT=3 SINT=3 TINT=2
GSURFACE 1 37 407 371 EL1=18 EL2=5 NODES=9 NF=2 EF=1
THICKNESS 1 0.27
STRESSTABLE 1 1 2 3 4
EDATA
ENTRIES EL NTH TABLE
1 1 1
TO
90 1 1
LIST ENODES 1 90
****************
* STEEL BOXES *
****************
EGROUP 2 SHELL M=2 RINT=3 SINT=3 TINT=2
COORDINATES
ENTRIES NODE X Y Z
408 -3.675 21.6 0.
445 -2.450 21.6 0.
482 -1.225 21.6 0.
519 1.225 21.6 0.
556 2.450 21.6 0.
593 3.675 21.6 0.
630 -3.675 21.6 1.25
667 -1.225 21.6 1.25
704 1.225 21.6 1.25
741 3.675 21.6 1.25
```

```
NGENERATION TIMES=36 NSTEP=1 YSTEP=-0.6
408 445 482 519 556 593 630 667 704 741
ENODES
ENTRIES EL N1 N2 N3 N4 N5 N6 N7 N8 N13
1 408 410 484 482 409 447 483 445 446
19 519 521 595 593 520 558 594 556 557
37 75 77 410 408 76 632 409 630 631
55 149 151 484 482 150 669 483 667 668
73 223 225 521 519 224 706 520 704 705
91 297 299 595 593 298 743 594 741 742
EGENERATION TIMES=17 ESTEP=1 NSTEP1=2
1 19 37 55 73 91
THICKNESS 2 0.012
THICKNESS 3 0.025
STRESSTABLE 1 1 2 3 4
EDATA
ENTRIES EL NTH TABLE
37 2 1
TO
108 2 1
1 3 1
TO
36 3 1
LIST ENODES 1 108
**************
* DIAPHRAGMS *
**************
EGROUP 3 SHELL M=2 RINT=3 SINT=3 TINT=2
COORDINATES
ENTRIES NODE X Y Z
778 -2.45 21.6 1.25
779 2.45 21.6 1.25
780 -2.45 0.0 1.25
781 2.45 0.0 1.25
ENODES
ENTRIES EL N1 N2 N3 N4 N5 N6 N7 N8 N13
1 149 75 408 482 112 630 445 667 778
2 297 223 519 593 260 704 556 741 779
3 185 111 444 518 148 666 481 703 780
4 333 259 555 629 296 740 592 777 781
THICKNESS 1 .010
STRESSTABLE 1 1
EDATA
ENTRIES EL NTH TABLE
1 1 1
TO
4 1 1
LIST ENODES 1 4
BOUNDARIES IDOF=111000
444 481 518 555 592 629
BOUNDARIES IDOF=101000
408 445 482 519 556 593
```

```
****************
* LOAD APPLIED *
****************
LOADS CONCENTRATED
75 3 71150. 1 0
STEP 2 TO
101 3 71150. 1 0.936
149 3 152960. 1 0.
STEP 2 TO
175 3 152960. 1 0.936
223 3 -10200. 1 0.
STEP 2 TO
249 3 -10200. 1 0.936
297 3 1620. 1 0.
STEP 2 TO
323 3 1620. 1 0.936
85 3 101640. 1 0.
STEP 2 TO
111 3 101640. 1 0.936
159 3 218500. 1 0.
STEP 2 TO
185 3 218500. 1 :936
233 3 -14580. 1 0.
STEP 2 TO
259 3 -14580. 1 0.936
307 3 2320. 1 0.
STEP 2 TO
333 3 2320. 1 0.936
LIST LOADS CONCEN 75 333
FRAME
MESH
ADINA OPTIM=YES
LIST COORD 1 781
END
```

Appendix B

Some Graphs Representing the Variation of Acceleration With Time

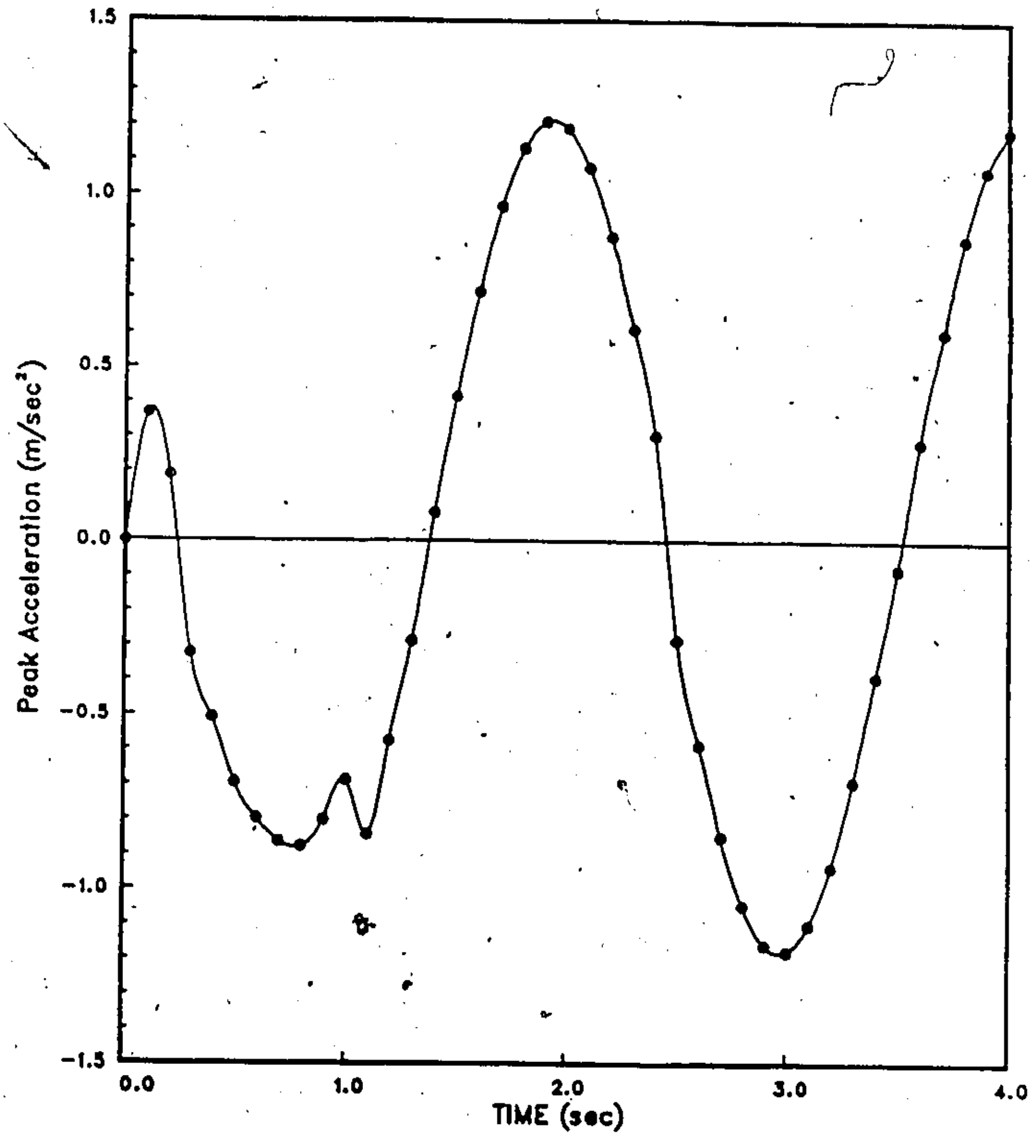

Figure B.1: Truck Velocity = 60 Km/H (Load at centre of lane)

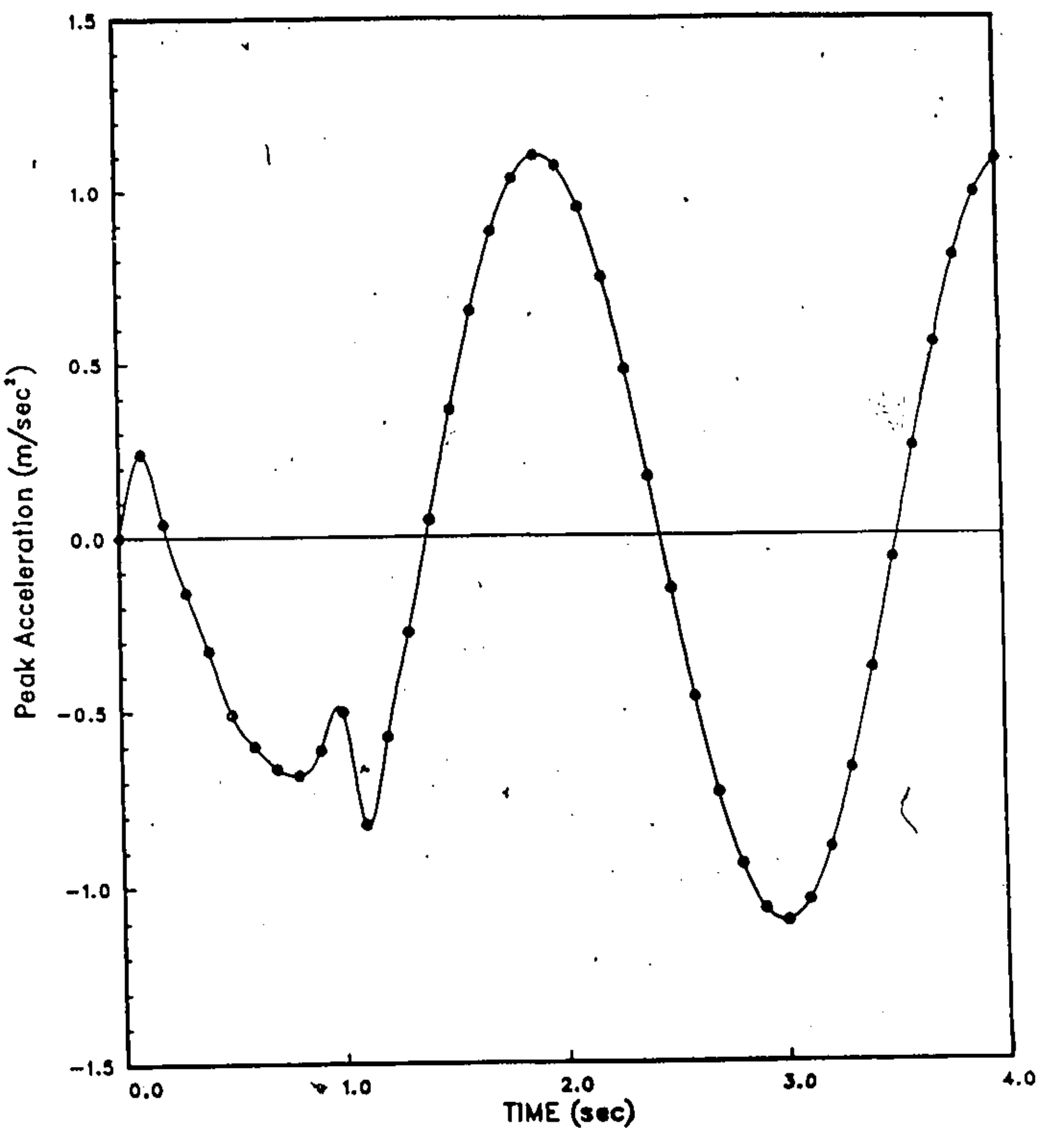

Figure B.2: Truck Velocity = 60 Km/H (Load at Middle of the Bridge)

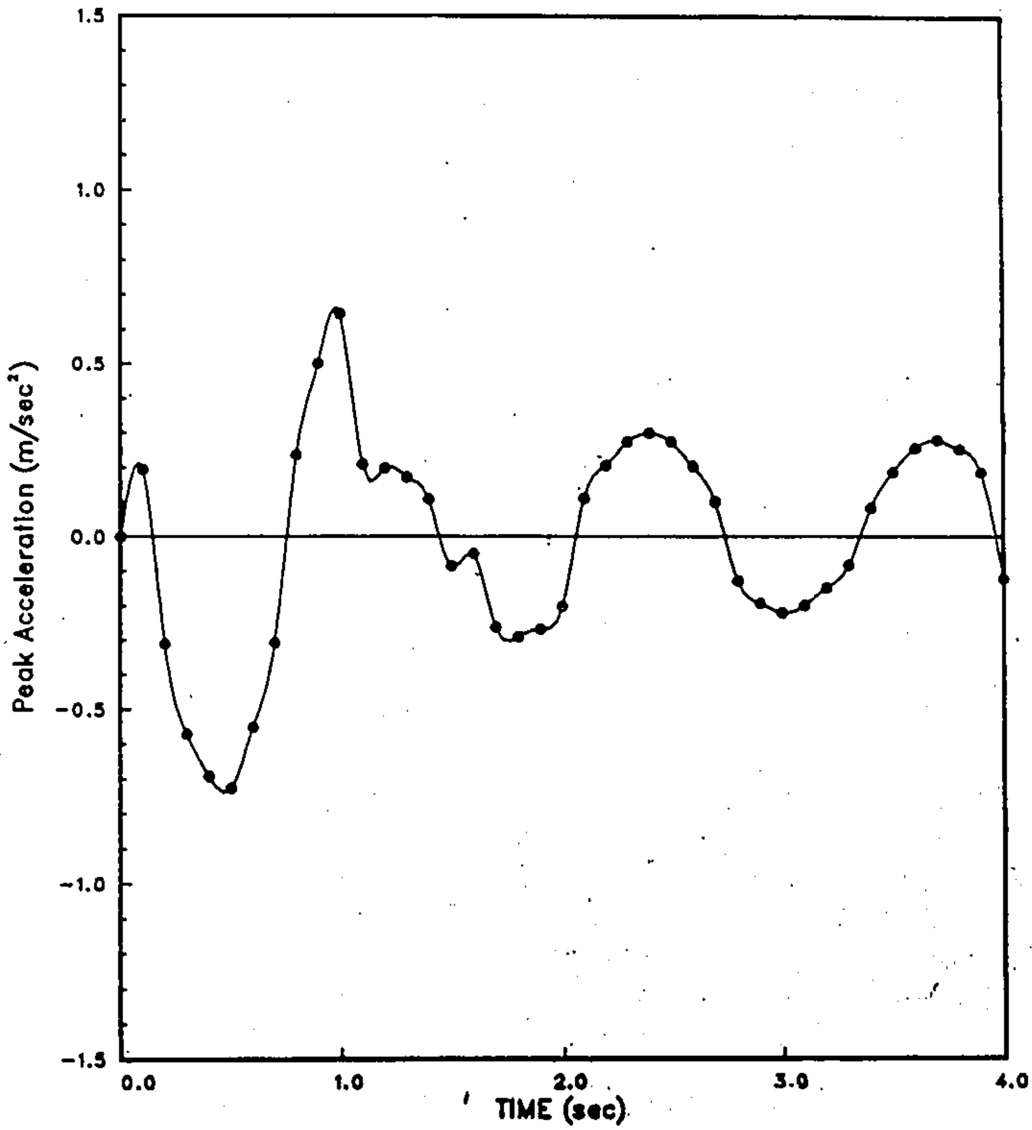

Figure B.3: Truck Velocity = 60 Km/H diaph = 10 mm (Load at centre of lane)

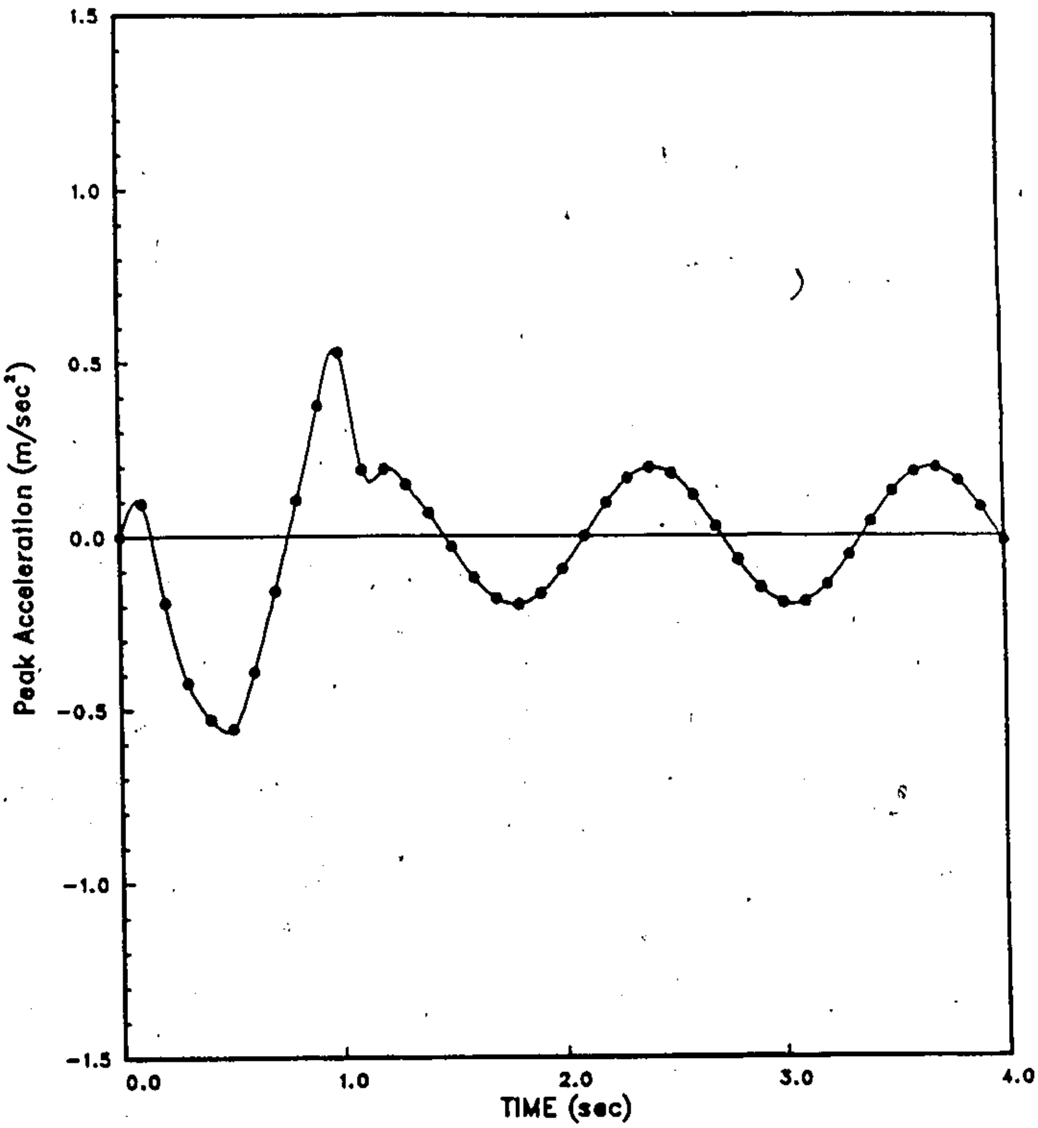

Figure B.4: Truck Velocity = 60 Km/H diaph = 10 mm (Load at Middle of the Bridge)

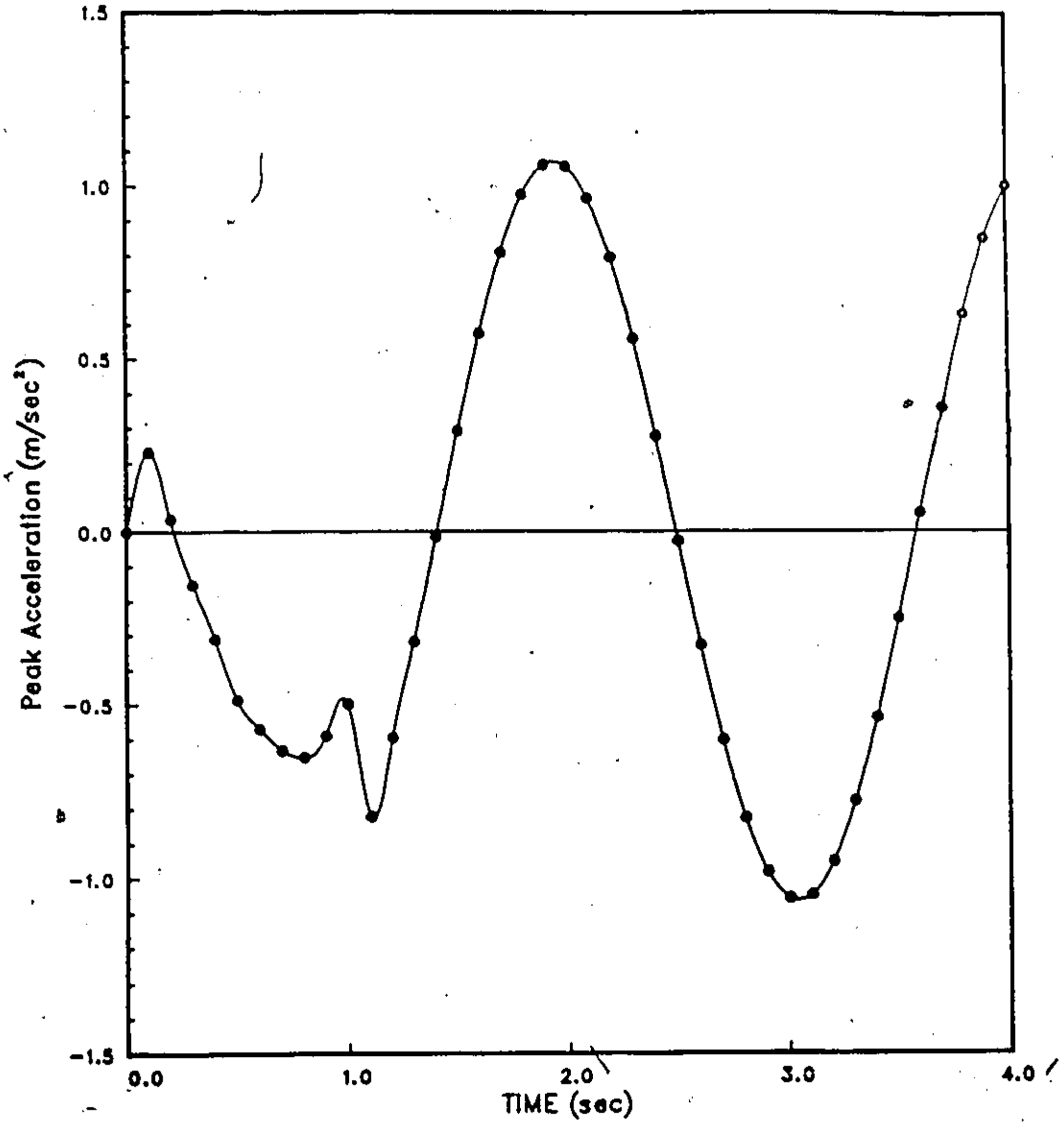

Figure B.5: Truck Velocity = 60 Km/H with 5 Bracings (Load at centre of lane)

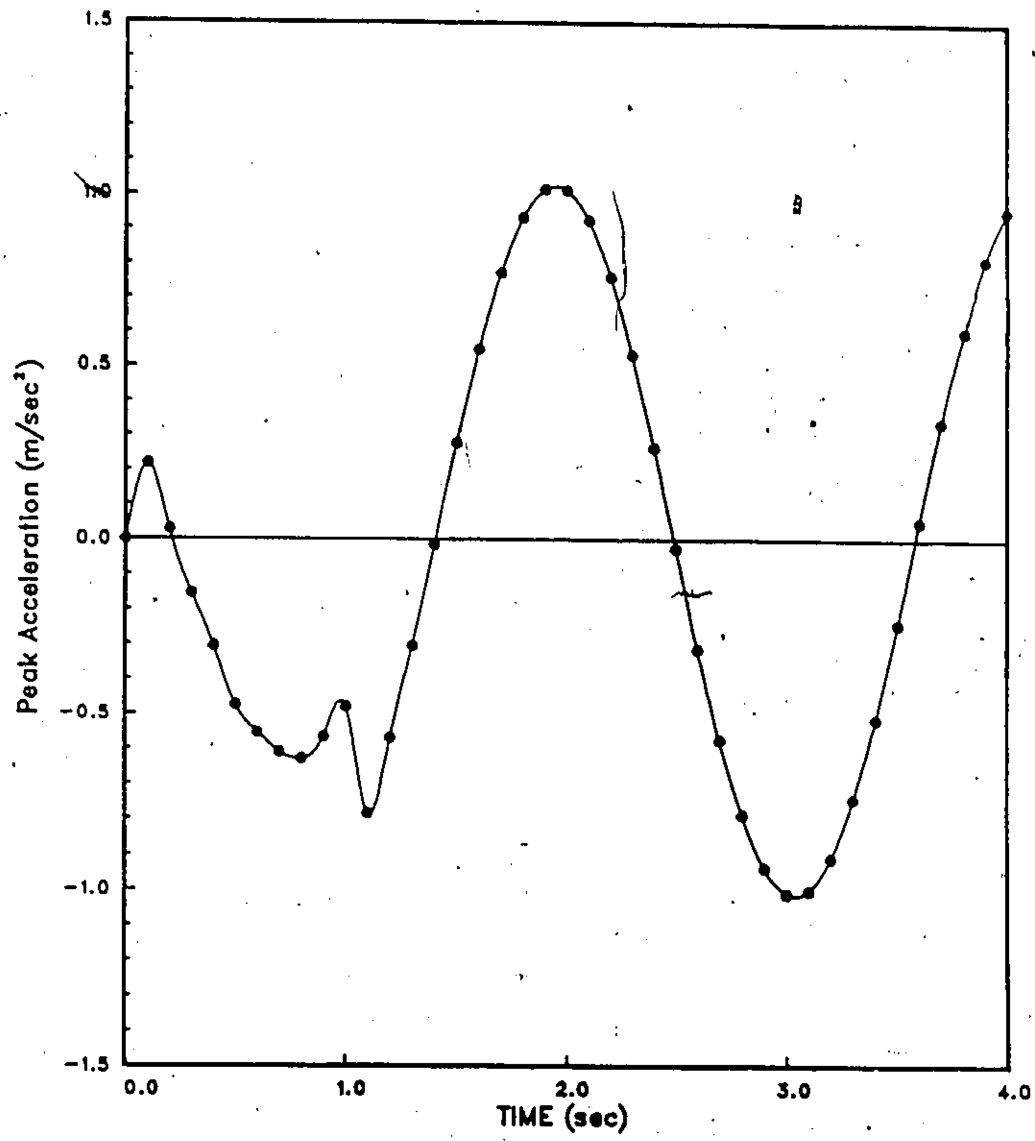

Figure B.6: Truck Velocity = 60 Km/H with 5 Bracings (Load at Middle of the Bridge)