DYNAMIC ANALYSIS OF RAILWAY STEEL ARCH BRIDGE

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work being presented in the dissertation entitled "**DYNAMIC ANALYSIS OF RAILWAY STEEL ARCH BRIDGE**" towards partial fulfillment of the requirements for the award of the degree of **Master of Technology** in **Earthquake Engineering** with specialization in **Structural Dynamics**, submitted to Earthquake Engineering Department, Indian Institute of Technology Roorkee, Roorkee, is an authentic record of my own work carried out from July 2005 to June 2006, under the guidance of **Shri R.N. Dubey,** Assistant Professor, Department of Earthquake Engineering, IIT Roorkee.

The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

Date: 30.6.06

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CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

An arch bridge is a bridge with abutments at each end shaped as a curved arch. These bridges work by transferring the weight of the bridge and its loads partially into a horizontal thrust against abutments at either side. In this dissertation, two arch bridges, one with two ribs and another with three ribs have been modeled as three-dimensional structures with overall span length 225m and rise 50m.The Bridge, is trussed arch with ribs as steel boxes. Both of them are analyzed for dynamic loading. Due to seismic zone V and high wind speed and rocky type soil with 69m deep gorge, arch bridge has been taken for analysis. The end conditions are taken as one side fixed and one having universal ball joint. As the railway bridge is being analyzed, proper train loading for broad gauge has been considered. Wind load distribution for open structure has been done for trussed arch bridge. The bridge is open structure and accordingly proper distribution of wind load has been taken into account. The analysis has been done using STAAD Pro 2004. As there is truss arch system no moment and shear forces will act on the structure. The results have been interpreted by analyzing time period, participation factor, deflections and mode shapes. Arch bridge with two ribs shows more deflection in transverse direction, which is on higher side where as the arch bridge with three ribs is more safe to lateral and vertical deflections.

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CHAPTER 1

INTRODUCTION

An arch may be defined as a member, shaped and supported in such a way that the intermediate transverse loads are transmitted to the supports primarily by axial compressive forces in the arch. The arch form is intended to reduce bending moments in the superstructure and is economical in material, compared with an equivalent straight, simply supported girder or truss. The horizontal thrust is resisted by the foundation or by a girder or truss running longitudinally beneath the deck for the full length of the span. Figure 1.1 shows various components of a typical arch bridge. Steel arch bridges are generally used to support either highways or railways. The typical span for steel arches ranges from 50 - 300 meters.

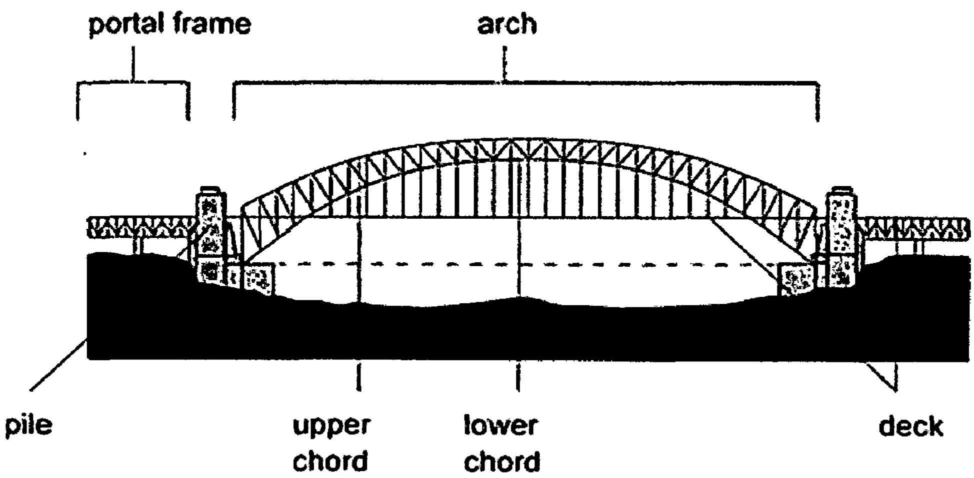

Figure 1.1 Typical Components of an Arch Bridge

1.1 Types of Arch Bridges: The nomenclature of the structural elements of an arch bridge depends on end conditions and hinges. Structurally there are four basic arch types: hinge-less, two-hinged, three hinged and tied arches.

Hinge-less Arch Bridge: The hinge-less arch uses no hinges and allows no rotation at the foundations as given in Figure 1.2. As a result a great deal of force is generated at the foundation (horizontal, vertical, and bending forces) and the hinge-less arch can only be built where the ground is very stable. However, the hinge-less arch is a very stiff structure and suffers less deflection than other arches.

Two Hinged Arch Bridge: The two-hinged arch uses hinged bearings, which allow rotation as shown in Figure1.3. The forces generated at the bearings are horizontal and vertical forces. This is perhaps the most commonly used variation for steel arches and is generally a very economical design.

Three Hinged Arch Bridge: The three-hinged arch as shown in Figure 1.4 adds an additional hinge at the top or crown of the arch. The three-hinged arch suffers very little if there is movement in either foundation (due to earthquakes, sinking, etc.) However, the three-hinged arch experiences much more deflection and the hinges are difficult to fabricate. The three-hinged arch is rarely used anymore.

The Tied Arch Bridge: The tied arch as shown in Figure 1.5 is a variation on the arch, which allows construction even if the ground is not solid enough to deal with the horizontal forces. Rather than relying on the foundation to restrain the horizontal forces, the girder itself "ties" both ends of the arch together.

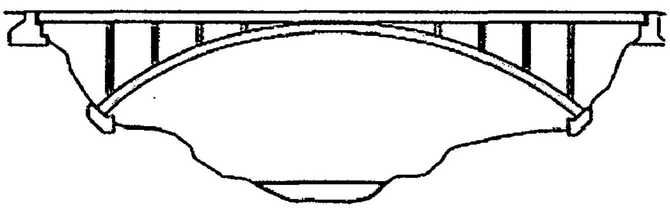

Figure 1.2 Hinge less Arch

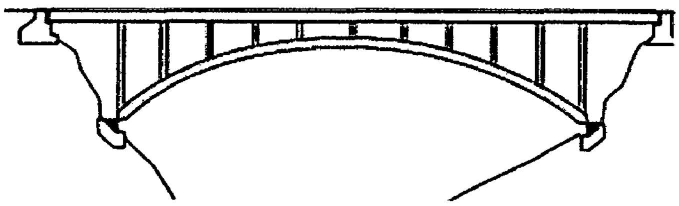

Figure 1.3 Two Hinged Arch

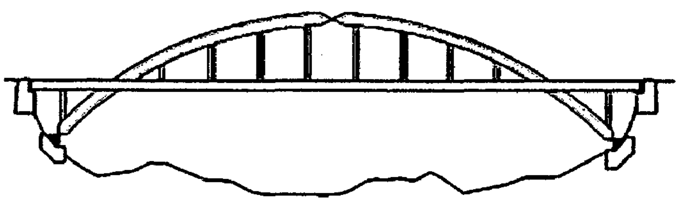

Figure1.4 Three Hinged Arch

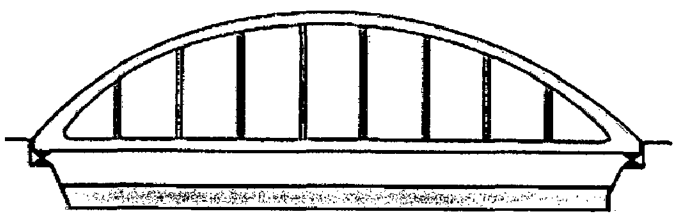

Figure 1.5 Tied Arch

1.2 Arch Parameters: The parameters of the arch can be grouped as geometrical and vibrational. Geometrical parameters are those that deal with the geometry of an arch. The main dimensions of arch are combined in the form of two non –dimensional parameters i.e.

ratio of rise to span = f/L and ratio of span to radius of gyration of arch cross-section at the crown $= L/k$. These two parameters are necessary and sufficient to describe uniform circular arches. However, for non-uniform, non circular arches, two additional parameters i.e. shape factor s equal to ratio of drop of arch axis at quarter point to rise f and form factor F, which characterizes the variation of the cross-section of the arch rib are required. For practical arches the coefficient 's' may vary from 0.15 to 0.25 and coefficient F may vary from 0.15 to 0.40. 's' is equal to 0.25 corresponds to parabolic arch.

where,

f = Rise of arch

L = Length of span

k = Radius of gyration

s = Shape factor

Vibrational parameters are those that deal with dynamic characteristics of an arch. Following relation expresses the period of vibration in rth mode of vibration of an arch of any shape, form or boundary condition:

$$T_r = C\left(\frac{f}{L}, \frac{L}{k}, F, S, r\right) L \sqrt{q/_{Eg}}$$

where,

C = A coefficient which is a function of the quantities in the parentheses

q =Weight density of arch material

E = Modulus of elasticity of arch material

g = Acceleration due to gravity

Besides, the parameters $f/_L$, $L/_k$, F and s. The other parameters representing the dynamic characteristics are fundamental period of arch, T, Mass distribution along the curved length of arch, damping in different modes of vibration and ground motion represented by the accelerogram $\ddot{x}(t)$ and $\ddot{y}(t)$.

1.3 Range of parameters

1. Rise span ratio $f/_L$: The rise to span ratio of arches chosen in study are based on practical considerations. The most common values adopted in practice vary from 0.15 to 0.25.

2. Slenderness ratio L/k : The L/k ratio commonly varies between 200 and 400; the lower values apply to bridge arches and the values greater than 300 are applicable for hangar arches. This factor affects the rib shortening and buckling in arch.
3. Shape factor: Generally the shapes taken are circular, parabolic and non-uniform.
4. Form factor: Generally three values of form factor 'F' are used i.e. 0.15, 0.25and 0.40. Form factor help to know thickness of arch rib.
5. Fundamental period of arch

   The fundamental time periods for arches in hangars and auditoria having spans less than 100m fall in the range of about 1 to 2 seconds and for bridge arches having spans less than 60 m lies in between 0.5 to 1 second. For common arches it is 0.5 to 3 seconds.
6. Mass distribution in arches. Three types mass distribution are considered:
   a) Self weight distributed along curved length
   b) Additional mass distributed uniformly along the span besides the own weight of arch, like that in open spandrel arch bridges, and
   c) Additional mass varying from minimum at the crown to maximum at the springing like that in filled spandrel arch bridges.
7. Damping could be due to several causes, such as, internal friction, air damping, friction at joints etc. The most common way of considering damping in dynamic analysis is to consider viscous damping in different modes of vibration. The damping is considered as 2% critical damping, which is considered as uniform in all modes.
8. Ground Motion Characteristics: The influence of characteristics of earthquake ground motion upon the structural response is an important factor.
9. Bridge dynamic response characteristics: Independent of specific dynamic input, each bridge system is represented within elastic range by dynamic response modes referred to as the natural modes of vibration, characterized by independent mode shapes with corresponding period of vibration.
10. Single Degree of Freedom Characteristics. The fundamental or first mode of vibration characteristics can be found for simple systems.

1.4 Scope of the Work: After girders, arches are the second oldest bridge type and a classic structure. Arches are good choices for crossing valleys and rivers since the arch doesn't require piers in the center. Arches add to the aesthetics of bridge. Arches use a curved structure, which provides a high resistance to bending forces. Unlike girder and truss bridges, both ends of an arch are fixed in the horizontal direction (i.e. no horizontal movement is allowed in the bearing). Thus when a load is applied on the bridge (e.g. a train passes over it), horizontal forces occur in the bearings of the arch. These horizontal forces are unique to the arch, as a result arches can only be used where the ground or foundation is solid and stable. The site where Railway Bridge is to be constructed is in **Kashmir** where both seismic load and wind load are deciding factor for the analysis. Using the software STAAD Pro 2004 the analysis is to be done for railway bridge .The broad gauge modeling is to be done for the bridge with 225m spans and 50m rise. The three dimensional analysis is to be performed simultaneously for arch with two rib and three rib and the one within safe horizontal and vertical deflection is selected. The analysis is to be done on trussed arch with thick beam elements for meshing the ribs. The geometry of the bridge requires detailed structural analyses to investigate their behavior under different loading conditions and also safe analysis.

1.5 Organization of the Dissertation

Chapter 1 deals with the basic types of arch bridges and their behaviors, describing the geometrical and vibrational parameters of arches that play a vital role and also ranges of these parameters. Chapter 2 deals with the literature review related to dynamic analysis of railway arch bridges. Chapter 3 deals with dynamic analysis of bridge. The various methods of dynamic analysis, assumptions and various parameters are described in case of bridges. Chapter 4 describes the loads to be considered in case of arch railway bridges. Chapter 5 gives the complete details about the project site, location and selection of bridges, what are the important features of the project site as well as of the bridge that is to be analyzed there. Chapter 6 deals with the results and analysis details of the proposed arch bridge. Chapter 7 is the conclusion of the work done. In this chapter every thing is tied together by presenting the important points of this study.

CHAPTER 2

LITERATURE REVIEW

The initial study and development about arch bridge dates to back late 1960s. The late 1960 and early 1980 shows more literature on railway arch bridges, dynamic analysis of arch bridges and other related topics. The successful designing and analysis of arch bridges for long spans without piers for static and dynamic loading for train in speed and for railway bridge modeling leads to progressive studies in this area. Later in 1980s, there are many literatures regarding dynamic analysis of arch bridges.

Chatterjee, et al [6], (1995) presented a dynamic analysis of arch bridges traversed by a single moving load using a mixed approach in which the advantages of continuum and lumped mass methods have been combined. A flat plate supported by struts of equal stiffness idealizes the bridge deck. The applicability of the method is studied by comparing the results with those obtained by the lumped mass method. The results indicate that for a relatively stiff arch supporting a flexible deck, the proposed and the lumped mass methods show good agreement.

Gorman, et al [25], (2005) obtained an accurate analytical type solution by utilizing the superposition method for the free vibration frequencies and mode shapes of multi-span bridge decks. In this approach to the problem, a separate solution is obtained for the lateral displacement of each individual span. Simple support is provided at the outer extremities of the deck. Conditions of plate continuity and zero lateral displacement are enforced at inter-span support locations. Edges running parallel to the long centre line of the bridge are free. Eigenvalues and mode shapes are presented for a typical bridge deck of three spans, though it is shown that decks of any number of spans may be analyzed.

Xia, et al [22], (2005) studied the dynamic interaction between high-speed train and bridge by theoretical analysis and field experiment. A computational model of train–bridge system is established. Each vehicle is described by 27 degrees of freedom. The bridge is modeled by modal superposition technique. The dynamic responses of the bridge such as dynamic deflections, lateral amplitudes, lateral and vertical accelerations, lateral pier amplitudes, and the vehicle responses such as derail factors, off load factors, wheel/rail forces and car–body accelerations are calculated.

Ching Jong Wang [27], (2005) studied that under strong ground excitations, highway bridge structures may experience severe nonlinear behaviors including the yielding and plastic deformation of pier members, and sometimes the pounding between adjacent decks induced by the local failure of hinge bearings necessary for restraining the girders. A kind of hinge bearing requires the use of steel dowels and is typical in many existing bridges. The objective of his study was to investigate why such hinge bearing did not function and how it contributed to the deck-falling failure of one particular highway bridge stricken by the 1999 Chi–Chi earthquake (magnitude 7.3) in Taiwan. He achieved this by incremental time history analysis for discrete dynamic systems on a group of models that incorporate sliding and impacting elements, to address nonlinear behaviors as a result of the failed bearing.

Binodi, et al [29], (2005) investigated the dynamic interaction between a running train, the track structure and the supporting bridge resorting to substructure technique. The train is idealized as a sequence of identical vehicles moving at constant speed. Both the rails and the bridge are modeled as Bernoulli–Euler beams, while the ballast is characterized as a viscoelastic foundation. A variant of the component-mode synthesis method is proposed to couple the continuous (rails and bridge) and discrete (train) substructures.

Fairfield et al [13], (1998) developed a method enabling the optimal design of arch bridges based on their modal characteristics. The relationship between the resonant frequencies, a function of mass and stiffness, and the load carrying capacity of the arches was investigated. Natural frequency and dynamic direct implicit time integration analyses were performed to determine the resonant frequencies and analyzed the responses to impact loading. Their collapse loads, under different loading regimes, were also investigated using elasto-plastic non-linear finite element analysis. The collapse loads were found to be related to the resonant frequencies and an optimal design could therefore be achieved. Under vertical loading, arches with span to rise ratios between 4 and 5 had both the highest resonant frequencies and collapse loads.

Memory et al [30], (1994) investigated about the natural frequencies and associated mode shapes of bridge superstructures. He compared field observations with theoretical idealizations and find that, while a single beam idealization is accurate for straight, non skewed bridges and for some continuous superstructures, many other bridges require an eigen value analysis of a finite beam element grillage. A simplified method for estimating the natural frequency of vibration is developed. An application of the Rayleigh method to a grillage model of the bridge is done and results were accurate to within 10%. They compared the effects of using the static and dynamic moduli of elasticity of concrete in estimating the natural frequency of vibration, and conclude that the dynamic modulus is more appropriate.

Kim et al [28], (2005) proposed a three-dimensional means of analysis for the bridge–vehicle interaction to investigate the dynamic responses of a steel girder bridge and vehicles. A cargo truck, dump truck and steel girder bridge are considered numerical models and measured roadway roughness profiles are used for analyses. The analytical dynamic wheel loads and acceleration responses of the heavy vehicles and responses of the bridge are compared with data from field tests to verify the validity of the proposed procedure.

Soyluk K [24], (2004) investigated the spatial variability effects of ground motions on the dynamic behavior of long-span bridges by a random vibration based spectral analysis approach and two response spectrum methods. The spatial variability of ground motions between the support points is taken into account with the coherency function, which arises from three sources: incoherence, wave-passage and site-response effects. Random vibration analyses are performed on two deck-type arch bridges and a cable-stayed bridge model. The results strongly imply that the filtered white noise ground motion model can be accepted as a rather convenient model to represent actual earthquake ground motions.

Fry´ba et al [21], (1999) described the static, dynamic and long-term tests of bridges in situ, which have been performed in the Czech and Slovak Republics since 1968. The standard methods are supplemented with the criteria for the elastic and permanent deformations, natural frequencies and the dynamic impact factors. The monitoring of stresses under usual traffic loads provides important data for the fatigue of bridges, for the estimation of their residual life and for the determination of inspection intervals. Modal analysis and identification ascertain the characteristic properties of bridges from their response. The damage in bridges may be reflected in the changes of their natural frequencies or modes of natural vibration. The vertical deflection that he got provides decisive values for static tests, while the comparison of calculated and measured natural frequencies is recommended after dynamic tests. The long-term experiments (monitoring) provide the data for fatigue assessment and for the estimation of inspection intervals.

CHAPTER 3

DYNAMIC ANALYSIS OF BRIDGE

Introduction

All real physical structures, when subjected to loads or displacements behave dynamically. The additional inertia forces from Newton's second law are equal to the mass times the acceleration. If the loads or displacements are applied very slowly then the inertia forces can be neglected and a static load analysis can be justified. Hence, dynamic analysis is a simple extension of static analysis. In addition, all real structures potentially have an infinite number of displacements. Therefore, the most critical phase of a structural analysis is to create a computer model, with a finite number of mass less members and a finite number of node (joint) displacements that will simulate the behavior of the real structure where the mass of a structural system can be assumed to be accurately lumped at the nodes. Also for linear elastic structures, the stiffness properties of the members with the aid of experimental data can be approximated with a high degree of confidence. Whereas, in dynamic loading, energy dissipation properties and boundary (foundation) conditions for many structures are difficult to estimate. This is always true for the cases of seismic input or wind loads. Therefore dynamic analysis is very important for such type of structure.

3.1 Dynamic Equilibrium

The force equilibrium of a multi-degree-of-freedom lumped mass system as a function of time can be expressed by the following relationship

$$F(t)_I + F(t)_D + F(t)_D = F(t) \quad \ldots\ldots(1)$$

where the force vectors at time t are

$F(t)_I$ is a vector of inertia forces acting on the node masses

$F(t)_D$ is a vector of viscous damping, or energy dissipation, forces

$F(t)_S$ is a vector of internal forces carried by the structure

$F(t)$ is a vector of externally applied loads

Above equation is based on physical laws and is valid for both linear and nonlinear systems if equilibrium is formulated with respect to the deformed geometry of the structure. For many structural systems, the approximation of linear structural behavior is made in order to convert the physical equilibrium statement, Equation (1), to the following set of second order, linear, differential equations

$$Mu''(t)_a + Cu'(t)_a + Ku(t)_a = F(t) \quad \text{.......(2)}$$

where, M is the mass matrix (lumped or consistent), C is a viscous damping matrix (which is normally selected to approximate energy dissipation in the real structure) and K is the static stiffness matrix for the system of structural elements. The time-dependent vectors $u(t)_a, u'(t)_a$ and $u''(t)_a$ is the absolute node displacements, velocities and accelerations, respectively.

3.2 Fundamentals of Dynamic Equilibrium

For seismic loading, the external loading $F(t)$ is equal to zero. The basic seismic motions are the three components of free-field ground displacements $u(t)_{ig}$ that are known at some point below the foundation level of the structure. Therefore, Equation (2) can be written in terms of the displacements $u(t)$, velocities $u'(t)$ and accelerations $u''(t)$ that are relative to the three components of free field ground displacements. Therefore, the absolute displacements, velocities and accelerations can be eliminated from Equation (2) by writing the following simple equations

$$\begin{aligned} u(t)_a &= u(t) + I_x u(t)_{xg} + I_Y u(t)_{yg} + I_z u(t)_{zg} \\ u'(t)_a &= u'(t) + I_x u'(t)_{xg} + I_y u'(t)_{yg} + I_z u'(t)_{zg} \quad \text{.......(3)} \\ u''(t)_a &= u''(t) + I_x u''(t)_{xg} + I_y u''(t)_{yg} + I_z u''(t)_{zg} \end{aligned}$$

where I_i is a vector where "i"is the subscript directional degrees-of-freedom which is unity in that particular direction and zero in all other positions. The substitution of Equation (3) into Equation (2) allows the node point equilibrium equations to be rewritten as

$$Mu''(t)+Cu'(t)+Ku(t)=-Mu''(t)_{xg}-Mu''(t)_{yg}-Mu''(t)_{zg} \quad \text{.........} (4)$$

where $MI = MI_i$. The simplified form of Equation (4) is possible since the rigid body velocities and displacements associated with the base motions cause no additional damping or structural forces to be developed. There are several different classical methods that can be used for the solution of Equation (4). Each method has advantages and disadvantages that depend on the type of structure and loading.

3.2.1 Step-By-Step Solution Method

The most general solution method for dynamic analysis is an incremental method in which the equilibrium equations are solved at times $Dt, 2Dt, 3Dt$ etc. There are a large number of different incremental solution methods. In general, they involve a solution of the complete set of equilibrium equations at each time increment. In the case of nonlinear analysis, it may be necessary to reform the stiffness matrix for the complete structural system for each time step. Also, iteration may be required within each time increment to satisfy equilibrium. As a result of the large computational requirements it can take a significant amount of time to solve structural systems with just a few hundred degrees-of-freedom. In addition, artificial or numerical damping must be added to most incremental solution methods in order to obtain stable solutions. For some nonlinear structures, subjected to seismic motions, incremental solution methods are necessary.

3.2.2 Mode Superposition Method

The most common and effective approach for seismic analysis of linear structural systems is the mode superposition method. This method, after a set of orthogonal vectors are evaluated, reduces the large set of global equilibrium equations to a relatively small number of uncoupled second order differential equations. The numerical solution of these equations involves greatly reduced computational time. It has been shown that seismic motions excite only the lower frequencies of the structure. Typically, earthquake ground accelerations are recorded at increments of 200 points per second. Therefore, the basic loading data does not contain information over 50 cycles per second. Hence, neglecting the higher frequencies and mode shapes of the system normally does not introduce errors.

3.2.3 Response Spectra Analysis

The basic mode superposition method, which is restricted to linearly elastic analysis, produces the complete time history response of joint displacements and member forces due to a specific ground motion loading. There are two major disadvantages of using this approach. First, the method produces a large amount of output information that can require an enormous amount of computational effort to conduct all possible design checks as a function of time. Second, the analysis must be repeated for several different earthquake motions in order to assure that all the significant modes are excited, since a response spectrum for one earthquake, in a specified direction, is not a smooth function. There are significant computational advantages in using the response spectra method of seismic analysis for prediction of displacements and member forces in structural systems. This method involves the calculation of only the maximum values of the displacements and member forces in each mode using smooth design spectra that are the average of several earthquake motions. In addition, it will be shown that the SRSS (square root of the sum of squares) and CQC (complete quadratic combination) methods of combining results from orthogonal earthquake motions will allow one dynamic analysis to produce design forces for all members in the structure.

3.3 Methods of Solution: The various methods to solve the solution of the equation that obtained from dynamic consideration are solved by linear approach in frequency domain.

3.3.1 Solution in Frequency Domain

The basic approach, used to solve the dynamic equilibrium equations in the frequency domain, is to expand the external loads $F(t)$ in terms of Fourier series or Fourier integrals. Therefore, it is very effective for periodic types of loads such as mechanical vibrations, acoustics, sea-waves and wind. However, the use of the frequency domain solution method for solving structures subjected to earthquake motions has the following disadvantages:

1. The mathematics is difficult to understand. Also, the solutions are difficult to verify.
2. Earthquake loading is not periodic; therefore, it is necessary to select a long time period in order that the solution from a finite length earthquake is completely damped out prior to the application of the same earthquake at the start of the next period of loading.
3. For seismic type loading the method is not numerically efficient. The transformation of the result from the frequency domain to the time domain, even with the use of Fast Fourier Transformation methods, requires a significant amount of computational effort.
4. The method is restricted to the solution of linear structural systems.

3.3.2 Solution of Linear Equations

The step-by-step solution of the dynamic equilibrium equations, the solution in the frequency domain, and the evaluation of eigenvectors require the solution of linear equations of the following form:

$$AX = B \qquad (5)$$

Where A is a $N \times N$ symmetric matrix, which contains a large number of zero terms. The $N \times M$ matrix, X displacement and B load matrix indicates that more than one load condition can be solved at the same time. Because the matrix is symmetric, it is only necessary to form and store the first nonzero term in each column down to the diagonal term in that column. Therefore, the sparse square matrix can be stored as a one-dimensional array along with an $N \times 1$ integer array that indicates the location of each diagonal term. If the stiffness matrix exceeds the high-speed memory capacity of the computer, a block storage form of the algorithm exists. Therefore, the capacity of the solution method is governed by the low speed disk capacity of the computer.

3.4 Undamped Harmonic Response

The most common and very simple type of dynamic loading is the application of steady-state harmonic loads of the following form:

$$F(t) = f \sin \varpi(t) \qquad (6)$$

The node point distribution of all static load patterns, f, which are not a function of time, and the frequency of the applied loading, ω, are user specified. Therefore, for the case of zero damping, the exact node point equilibrium equations for the structural system are

$$Mu''(t) + Ku(t) = f \sin \varpi(t) \qquad (7)$$

The exact steady-state solution of this equation requires that the node point displacements and accelerations be given by

$$u(t) = v \sin \varpi(t) \qquad (8)$$

Therefore, the harmonic node point response amplitude is given by the solution of the following set of linear equations:

$$\left[K - \varpi^2 M\right] v = f \qquad (9)$$

It is of interest to note that the normal solution for static loads is nothing more than a solution of this equation for zero frequency for all loads. It is apparent that the computational effort required for the calculation of undamped steady-state response is almost identical to that required by a static load analysis. The resulting node point displacements and member forces vary as $\sin(\varpi t)$. However, other types of loads that do not vary with time, such as dead loads, must be evaluated.

3.5 Undamped Free Vibrations

Most structures are in a continuous state of dynamic motion because of random loading such as wind, vibrating equipment, or human loads. These small ambient vibrations are normally near the natural frequencies of the structure and are terminated by energy dissipation in the real structure. However, special instruments attached to the structure can easily measure the motion. Ambient vibration field tests are often used to calibrate computer models of structures and their foundations. After all external loads are removed from the structure, the equilibrium equation, which governs the undamped free vibration of a typical displaced shape v, is

$$Mv'' + Kv = 0 \quad \text{..} (10)$$

At any time the displaced shape v may be a natural mode shape of the system, or any combination of the natural mode shapes. However, it is apparent the total energy within an undamped free vibrating system is a constant with respect to time. The sum of the kinetic energy and strain energy, at all points in time, is a constant and is defined as the mechanical energy E_M of the dynamic system and can be calculated from:

$$E_M = \frac{1}{2}v'^T Mv' + \frac{1}{2}v^T Kv \quad \text{.................................} (11)$$

3.6 Method of Analysis of Elastic Response

The dynamic response of a structure under earthquake motion is dependent upon the natural periods and mode shapes, damping characteristics and waveform of accelerogram. In order to obtain the elastic response of any structure under earthquake excitation two approaches are available (a) time wise superposition of response in various modes of vibration (b) direct integration of simultaneous differential equations of motion. The former approach has the merit because the first few modes have dominant contribution to the total response; only few equations are required to be integrated. But this approach requires a definite condition for damping matrix to be satisfied. While in second approach no pre-requisite on damping matrix is necessary but computational effort involved in integration of simultaneous differential equations is formidable.

3.6.1 Modes of free vibration of arches

Basically the arch has two kinds of modes of vibration in plane of structure,

Extensional mode: In this mode the arch axis is subjected to extensions and compressions as shown in Figure 3.1. Since the extensional stiffness is very much greater than the flexural stiffness, the period of vibration of this mode is short as compared to flexural modes.

Flexural mode: For symmetrical arches, flexural modes could be antisymmetrical or symmetrical. In these modes of vibration, bending deformations are significant as shown in Figure 3.2. For the common rise-span ratio of arches (1/6to1/4), the arch is more vulnerable to antisymmetrical deformation and therefore the antisymmetrical mode is the fundamental mode of vibration of arch for such case.

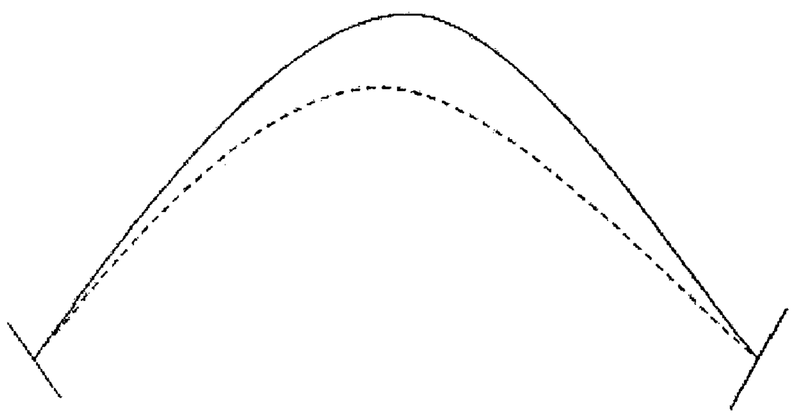

Figure 3.1 Extensional mode

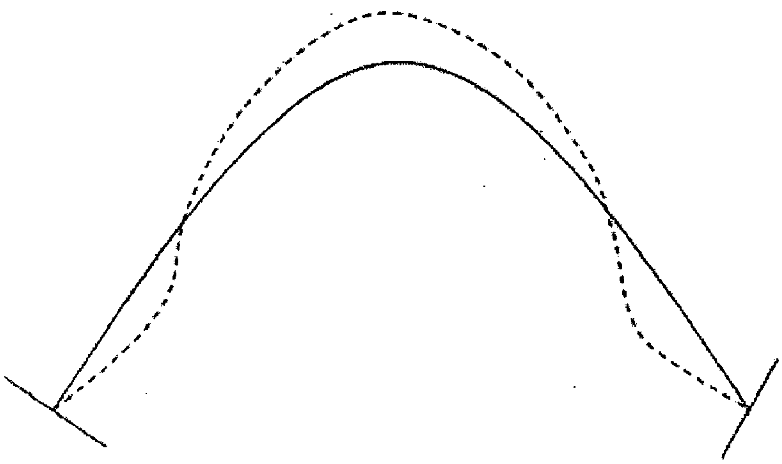

Figure 3.2 Flexural mode

3.7 Assumptions for Analysis

The following assumptions are made for the formulation of the problem.

1. The bridge deck is treated as a beam having uniform flexural rigidity and uniform mass per unit length for any segment of the beam between two struts; these two characteristics may change from segment to segment.

2. The vehicle is represented by a single constant vertical load P moving at a constant speed v along the centre line of the bridge deck so that a two-dimensional idealization of the bridge is possible.

3. The movement of the arch including the axial shortening of the struts with respect to time is assumed to be quasi-static, so that the deck vibration can be separated from the arch vibration.

The maximum deflection of Railway Bridge is dependent on speed of train, span length, mass, stiffness, damping of structures and axle load of train. Figure 3.3 shows the idealized arch bridge. Figure 3.4 shows the continuum model of arch bridge with lumped masses at interconnected springs. Here the mass is supposed to act at the joint of the structure. Figure 3.5 gives the continuum model of arch bridge with lumped mass distributed over small length of span. First modeling does the process of seismic bridge analysis. Then based on model, dimensions, section properties and material characteristics are approximated. After that the approaches like linear static analysis, linear elastic model, time history methods, are applied from which member forces, displacements and seismic forces are calculated. Flow chart in Figure 3.6 depicts the procedure.

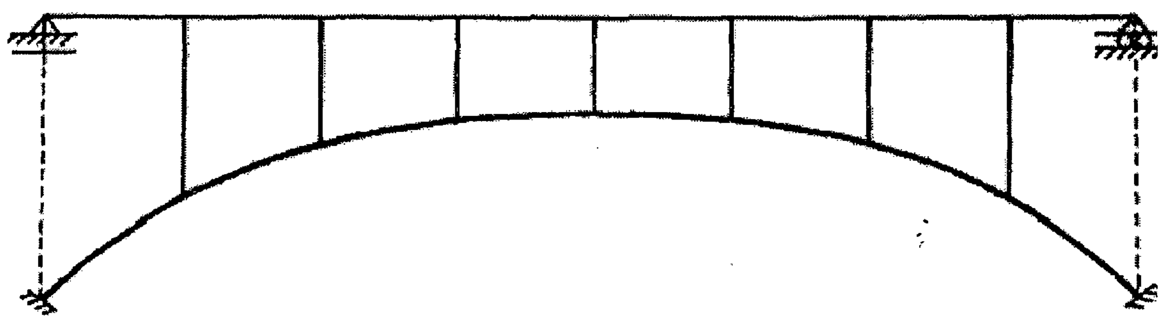

Figure 3.3 Idealized Arch Bridge

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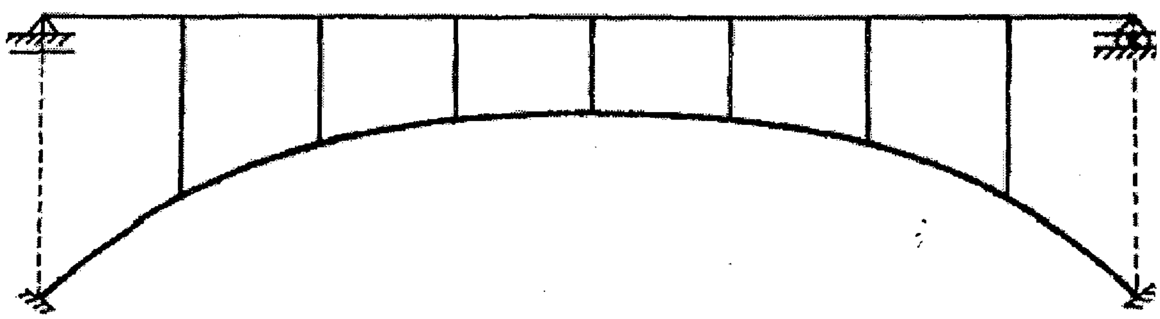

Figure 3.3 Idealized Arch Bridge

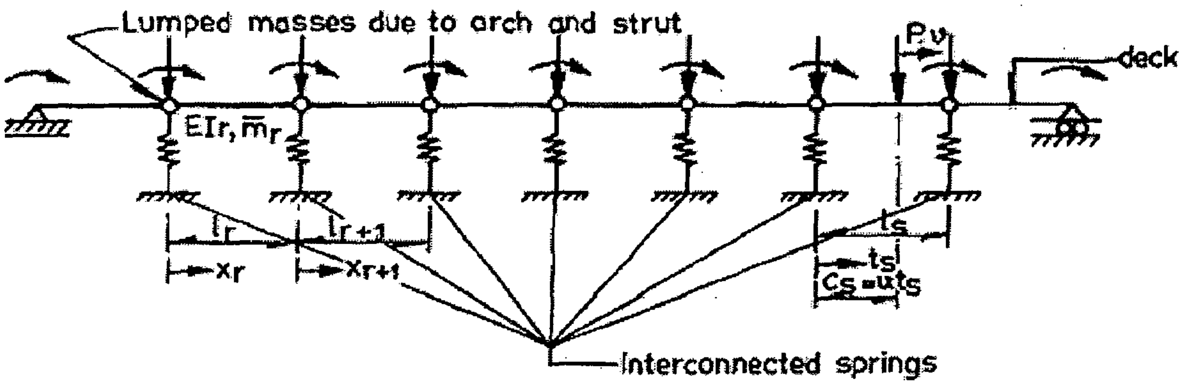

Figure 3.4 Continuum model of arch bridge with lumped masses

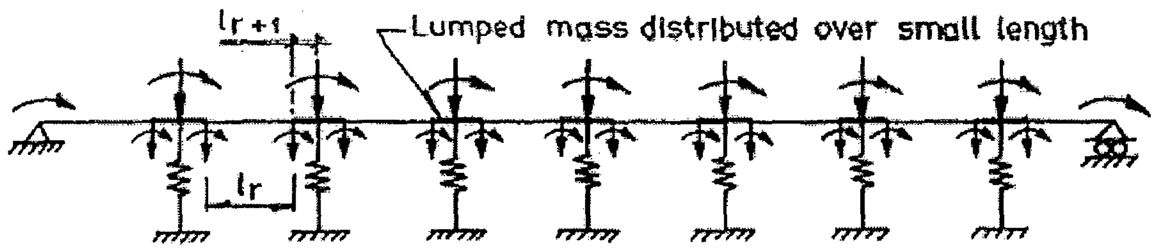

Figure 3.5 Continuum model of arch bridge with distributed masses

3.8 Parameters for Dynamic Analysis

The dynamic analysis of bridge requires certain parameters. Each of the deck and pier bodies of arch bridge is modeled as discrete mass supported by flexible pier column, through interfaces of bearings and gaps of joints. For a typical deck having mass M and damping C under the longitudinal ground acceleration $\ddot{X}_g$, the equation of motion can be expressed as

$$M\ddot{X} + C\dot{X} = -M\ddot{X}_g - F_i - F_s \quad (12)$$

F_i is the impact spring force initiated by a condition

$$X_2 - X_1 - G_P \rangle 0 \quad (13)$$

Where X_2 and X_1 respectively, represents displacements of two adjoining decks, G_P is the gap at expansion joints, F_S is the friction force at bearings governed by a condition

$$|X'_d - X'_P| \rangle 0 \quad (14)$$

Where X'_d and X'_p denote, respectively, velocities of a deck and its supporting pier.

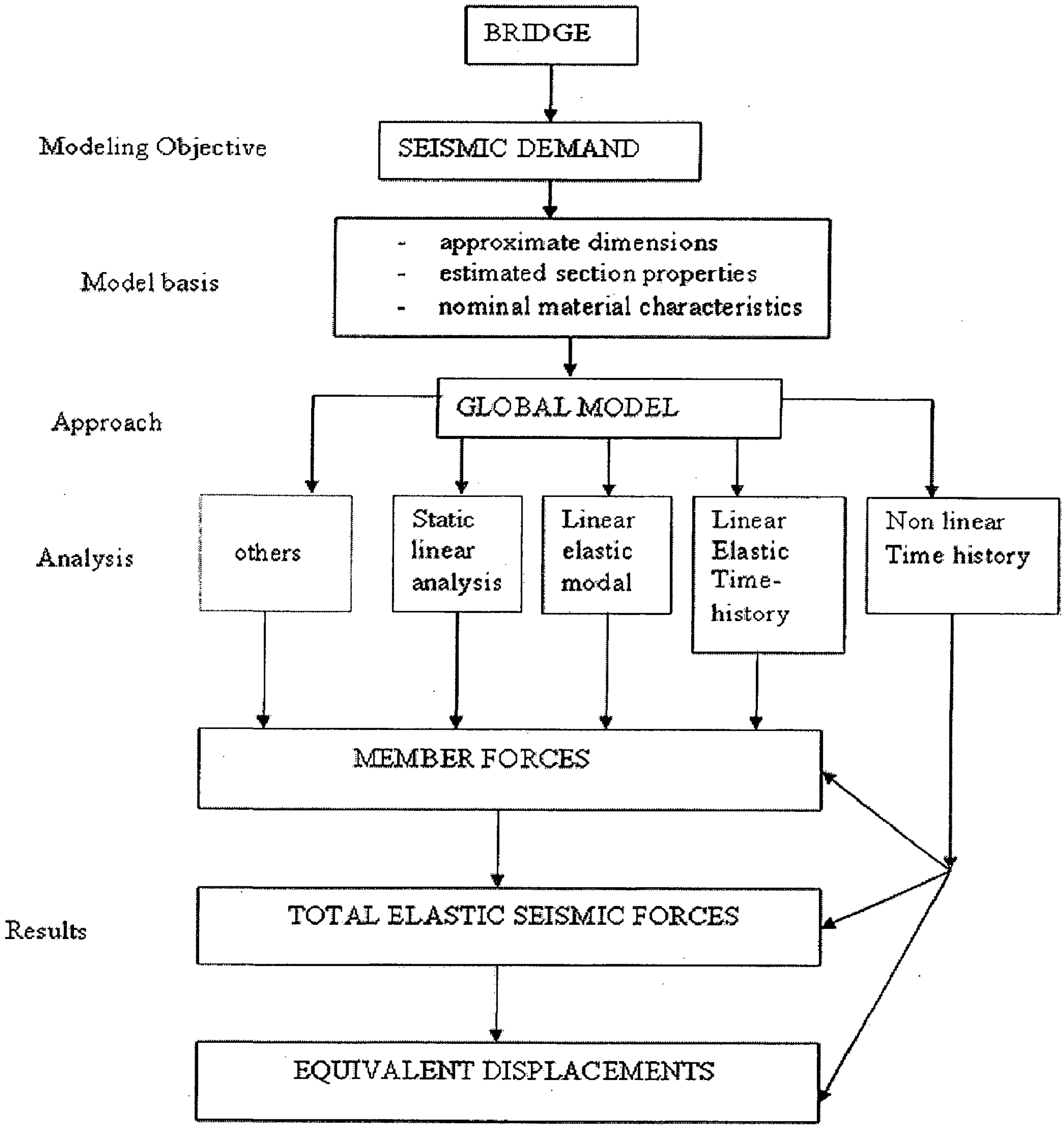

Figure 3.6 Flow Chart of Seismic Bridge Analysis Process

CHAPTER 4

LOADS ON ARCH

Introduction

The loads on arches are transferred outwardly by arching action and then to the foundations. For railway bridges the railway loading may be narrow, metre or broad gauge. Apart from this, dynamic loading for given seismic zone is considered. The loads due to erection, temperature differences, centrifugal actions, wind loads, tractive and braking forces are taken into consideration. The patterns in which these loads are to be applied play an important part.

4.1 Loading Pattern for Arch

Loading pattern for arch bridge is generally considered for full loading, for full loading over half the length of bridge, one side of the bridge fully loaded and alternates full loading over half the length of bridge.

Full Loading: In this case, the thrust H reaches a maximum, as shown in Figure 4.1. From computer calculations, it was found that the stiffening girder acting in bending contributes only 5% to the load carrying resistance of the bridge. As a result a very good estimate of the thrust can be obtained from the equation

$$H = {qL^2}/{8F} \quad \text{.....(1)}$$

where:

q is the uniformly distributed load

L is the span of the bridge

F is the rise of the arch, usually about $L/7$

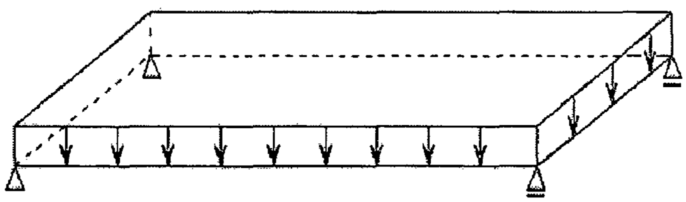

Figure 4.1 Full Loading

Full Loading over Half the Length of the Bridge: In this case load act only on half-length of bridge as shown in Figure 4.2. The half span of uniformly distributed loading 2q is equivalent to two superimposed loadings.

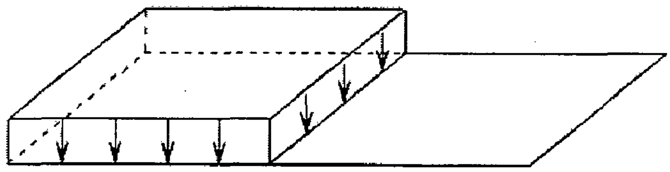

Figure 4.2 Full loading over half the length of the bridge

Figure 4.3 shows the total load 2q on half length of the bridge where it is taken as summation of full loading +q and anti symmetric loading as positive loading +q to the left and –q to the right.

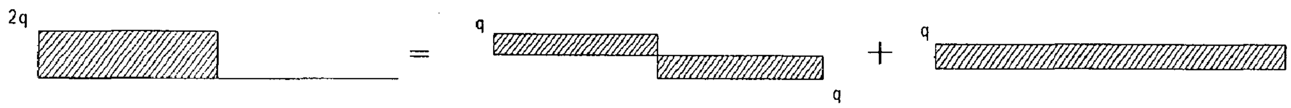

Figure 4.3 Half Span Loading as Combination of Symmetric and Anti Symmetric Load

Full loading primarily generates a thrust in the arch and a compensating tensile force in the girder. Due to the symmetry of the influence line, the second loading does not generate any thrust. As the deflection under this second loading is composed of two half waves, the girder can be considered to be composed of two parts with a "hinge" at midspan. This maximum bending moment occurs at L/4. The maximum bending moment is therefore approximately:

$$M_{\max} = qL^2/32 \qquad (2)$$

Full Loading on One Side of the Bridge: Again, in this case the equally distributed loading 2q can be considered as being composed of two superimposed loadings: Full loading +q on one side of the bridge, full loading -q on the other side of the bridge. Again the full loading primarily generates a thrust, inducing a tensile force in the girder, which is calculated from Equation (1). The one sided loadings tend to lozenge the bridge cross-section causing horizontal lateral forces on the arch and deck. These forces create horizontal bending moments in the girders and deck as shown in Figure 4.4.

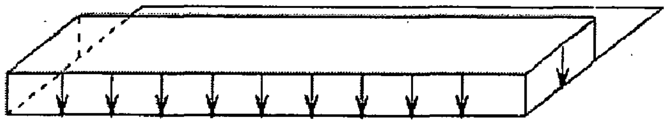

Figure 4.4 One side of bridge fully loaded

Alternating Full Loading over Half Length of the Bridge: Figure 4.5 shows alternating full loading over half the length of the bridge. Again the equally distributed loading 2q can be considered as being composed of two superimposed loadings: full loading: +q, to the left of one side of half the length of the bridge and full loading to the right of half the length of other side of the bridge. Again full loading primarily generates a thrust, inducing a tensile force in the girder, which is calculated from Equation (1).

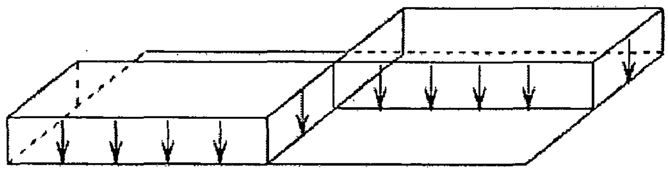

Figure 4.5 Alternating full loading over half the length of the bridge

4.2 Railway Design Loading

Dead Load: Superimposed dead loads on railway bridges usually include the rails, the sleepers, the ballast (or any other mean for transmission of train loads to the structural elements), other than this the self-weight of the structure is taken into consideration as dead load. Figure 4.6 shows dead load for railway loading.

Train Loads: Typical trainloads on bridges consist of a number of concentrated loads preceded and followed by a uniformly distributed load. Both loads are equally divided between the two rails. Figure 4.7 shows the trainload according to bridge rules1964 for broad gauge.

Longitudinal Tractive and Braking Forces: These forces are a percentage of trainloads that are considered as acting at rail level in a direction parallel to the tracks, as shown in Figure 4.8. As per bridge rules 1964, braking force of trainload is 20% of trainload. Tractive effort per loco for broad gauge is 50.0 tonnes.

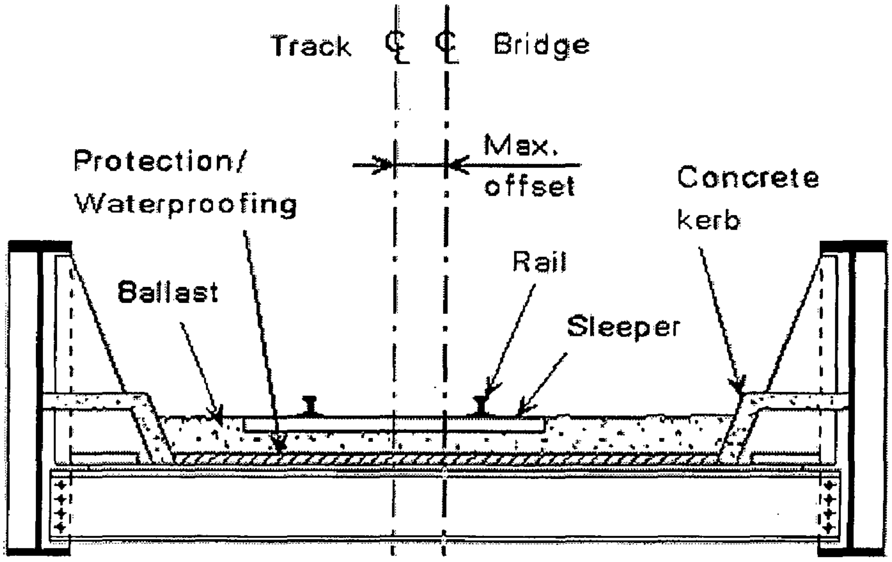

Figure 4.6 Dead Load

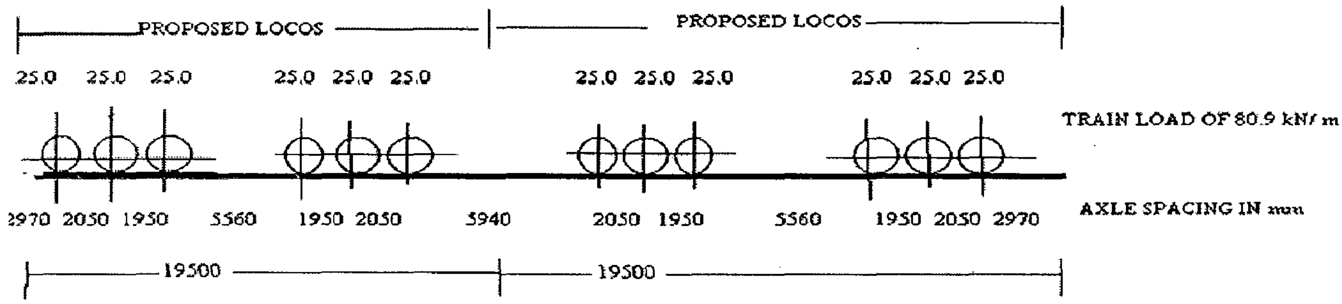

Figure 4.7 Modified Broad Gauge Loading (As per Bridge Rules 1964)

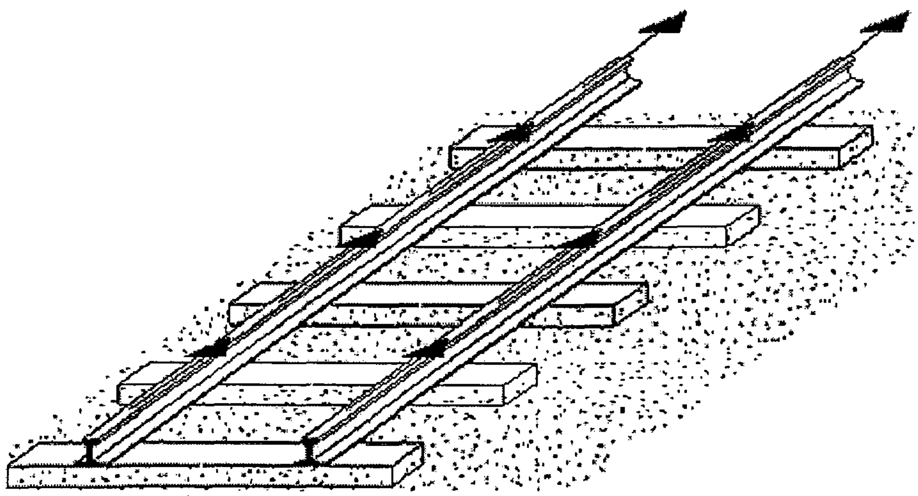

Figure 4.8 Longitudinal Tractive and Braking Forces

Dynamic Effects (Impact): Trainloads specified in the codes are equivalent static loadings and should be multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities. Values of dynamic factors depend on the type of deck (with ballast or open-deck) and on the vertical stiffness of the member being analyzed. For open-deck bridges values of dynamic factors are higher than for those with ballasted decks.

Centrifugal Forces: The nominal centrifugal load is applied corresponding to with the train loads and acts radially at a height of 1.83m above rail level for broad gauge and at 1.45m above rail level for meter gauge. As per IRS bridge rules 1964. Its value is obtained by

$$C = {WV^2}/{127R} \qquad \text{(In MKS Units)}$$

where,

C = Horizontal effect in KN/m of span.

W = Equivalent distributed live load in KN/m run

V = Maximum speed in km/hour,

R =Radius of the curve in m.

Wind Loads: The wind action on a bridge depends on site conditions and geometrical characteristics of the bridge. The maximum pressures are due to gusts that cause local and transient fluctuations about the mean wind pressure. Design gust pressures are derived from the design wind speed defined for a specified return period. The design wind load, are normally considered as horizontal loads and acting at the centroids of the exposed areas. For calculating design wind pressure as per IS: 875(Part3) 1987

Design wind velocity $V_Z = V_b \times k_1 \times k_2 \times k_3$

Design wind pressure $= 0.6V_{Z^2}$

where, V_Z = Design wind speed at any height in m/sec

V_b = Basic wind speed

k_1 =Probability factor (risk coefficient)

k_2 =Terrain, height and structure size factor

k_3 =Topography factor

Exposed area of traffic on bridges has the length corresponding to the maximum effects and in general a height of 2.50m above rail level in railway bridges.

Thermal Effects on Bridge Structures: Daily and seasonal fluctuations in air temperature cause two types of thermal actions on bridge structures, as referred in the code IRS Bridge rules: Changes in the overall temperature of the bridge (uniform thermal actions) and Differences in temperature (differential thermal actions) through the depth of the superstructure. The coefficient of thermal expansion for steel structures may be taken as 11.7 x 10^{-6} per$^{\circ}$C and for plain concrete it is taken as 10.8 x10^{-6} per $^{\circ}$C. These two types of thermal effect produce different types of response in a bridge. The overall change in temperature causes overall changes in bridge dimensions in an unrestrained structure (or so-called thermal stresses if these potential changes in dimension are resisted by the supports). Usually the structure is allowed to expand with minimal restraint by the provision of expansion joints and sliding bearings. The non-linear temperature distribution lead to self-equilibrating stresses on all cross-sections, even in unrestrained behavior as shown in Figure 4.9.

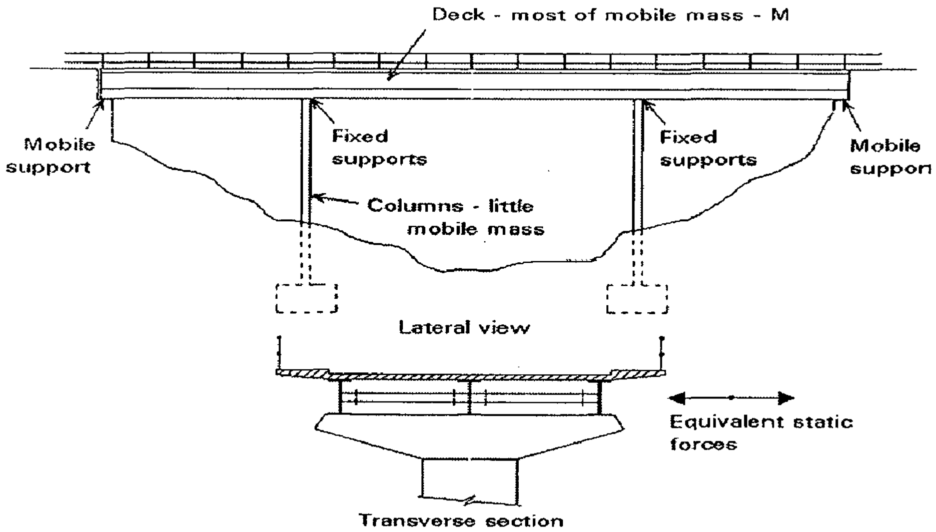

Figure 4.9 Bridge with Simple Dynamic Behavior

Earthquake Actions: Earthquake actions should be considered in bridge design. The behavior of a structure during an earthquake depends on its dynamic behavior, namely its natural vibration modes and frequencies, and damping coefficients. When the bridge has a simple dynamic behavior, for instance when the first vibration frequency is much lower than the other ones, the seismic action may be reduced to an equivalent static force.

Collisions: In structures where essential load-carrying elements may be subjected to impact by vehicles, ships or aircraft, the consequences should be taken into account by considering accidental load cases - unless the risk of such collisions is evaluated as being so small that it can be neglected. It is necessary in many cases to allow partial destruction or damage of the element that is directly hit.

Friction in Bearings have to be considered in the design of the structural elements. Modern sliding bearings are characterized by a coefficient of friction of approximately 0.03, if the sliding surfaces are absolutely clean. However, to take into consideration some deterioration in the sliding surfaces as well as tolerances in the positioning of the bearings it is recommended that the design is based on a typical coefficient of friction of 0.05. In a continuous beam with a fixed bearing at the center and longitudinally movable bearings on either side, expansion (or contraction) of the beam induces symmetrical frictional forces. These forces are in horizontal equilibrium if a constant coefficient of friction is assumed, and they normally result in moderate axial forces in the main girders.

4.3 Loads, forces and stresses

Loads and forces to be taken into account for the purpose of computing stresses, the following loads, where ever applicable should be taken into account in accordance with the requirements specified in the IRS Bridge Rules 1964: -

(a) Dead load

(b) Live load

(c) Impact effect

(d) Longitudinal forces

(e) Racking force

(f) Temperature effect

(g) Forces due to curvature and eccentricity of track

(h) Wind pressure effect

(i) Forces and effects due to earthquake

(j) Erection forces and effects

4.4 Combination of loads and forces

The worst possible load combination is that of dead load with live load, impact effect and forces due to curvature and eccentricity of track. When considering the member whose primary function is to resist longitudinal and racking forces due to live load, the term live load should be included with these forces. In normal cases, for bridges situated in seismic zones II to IV as per bridge rules, loads other than wind load and earthquake load form the worst possible combination. In zones where the intensity of traffic is high, the worst possible combination is of loads from (a) to (g) and (j) or (i). In case of bridges situated in seismic zone V, the worst possible combination is of loads from (a) to (g) and (h) or (i).

4.4.1 Primary and Secondary stresses: The primary stresses in the design of triangulated structures are defined as axial stresses in members calculated on the assumption that: all members are straight and free to rotate at the joints; all joints lie at the intersection of the centroidal axes of the members; all loads, including the weight of the members are applied at the joints.

4.4.2 Secondary stresses: Members are subjected not only to axial load as in primary stresses but also to bending and shear stresses. These stresses are secondary stresses. They are of two types: - stresses which are result of eccentricity of connections and of off joint loading generally (as loads rolling direct on chords, self weight of members and wind loads on members) and stresses which are the result of elastic deformation of the structure and the rigidity of the joints. They are called deformation stresses. Structures shall be designed, fabricated and erected in such a manner so as to minimize secondary stresses as far as possible.

CHAPTER 5

DETAILS OF PROJECT

Introduction

Construction of Jammu-Udhampur-Katra-Quazigund - Srinagar-Baramulla new rail link is the biggest project undertaken by the Indian Railways in the mountainous terrain since independence. Challenges in the construction of a Railway line through the hilly terrain start right from the conception stag itself. There are various constraints such as allowable maximum speed, high gradients, sharp curves, stations to be kept for optimum utilization, safety and minimum maintenance needed in future, in addition to the basic need for providing the link with the rest of the network. Projects in mountainous regions are associated with special features such as deep cuttings, high embankments, and tall piers and long span bridges across deep gorges and fast flowing flash flood rivers with big boulders and unusually long tunnels etc. These challenges are enhanced in view of the terrain in young Himalayas, where geology is poor and changes occur frequently. Surveys undertaken in the region have been a fascinating experience. The territory with virtually no habitation, no approach roads or even rudimentary pathways through dense jungles without any light or water connections, is a survey storey in itself. The tallest bridge is about 360 m above bed level and of a 505 m in length (single span) is also to be tackled in this reach over river Chenab. The project is a challenge to the Engineers of India in general and to the Railway Engineers in particular. Two more bridges on river Anji Khad and Pai Khad are to be constructed.

5.1 The Link

Indian railways are linking the Kashmir Valley with rest of the country by a rail link between Jammu and Baramulla. This project is perhaps the most difficult new railway line project undertaken on Indian subcontinent. The terrain passes through young Himalayas, which are full of geological surprises and continuous changes, due to the thrust region. The alignment of the project is as shown in Figure 5.1.

5.2 Survey

The alignment of Jammu - Udhampur – Katra – Quazigund – Baramulla rail link project passes through undulating terrain, especially in Katra – Quazigund section. Construction activities are already in full swing in Udhampur - Katra and Quazigund - Baramulla section. Beyond Katra (Km 30) up to Quazigund (Km 167), a part of alignment from 50 to 120 Kms falls in thick forest cover with no habitation and absence of trekking paths.

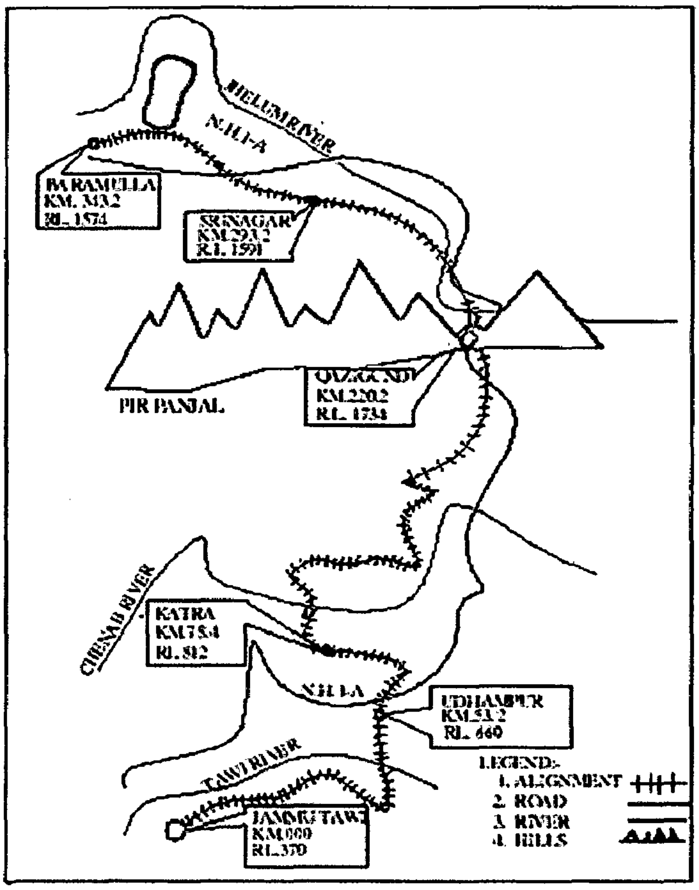

Figure 5.1 Alignment of Project

5.3 Bridges

In the hilly terrain, the construction of the bridges has been a difficult task and posed numerous challenges. Apart from the complexity of design, the construction of bridge requires great amount of planning and special techniques. The topography of the area resulted into long spans combined with large pier heights. The choice of alignment is most important for planning of bridges in hills. Geological features consisting of variable strata of sand rock, soft and hard shale, boulder-studded soil, etc. have also influenced the bridge lengths and span arrangements. The long spans were necessitated as a result of fixing pier locations in the middle of the gorges/streams so as to avoid constructing piers on sloping banks. This aspect itself called for arch bridge construction in bridges. This method has the added advantage of elimination of costly centering and false work and reduced requirement of shuttering and fast pace of construction. Detailed Design criteria were developed which were bridge specific.

The long spans and tall piers associated with highly seismic characteristics of the area have made the designs cumbersome and tricky. The bridges have been designed for Modified Broad Gauge (MBG) loading - 1987 as per Indian Railways Bridge Rules.

5.4 Seismic Design considerations

The bridge sites lie in the Seismic Zone V as per the current Seismic Zoning Map of India contained in IS: 1893-2002. The data show that seismic events having Richter's magnitude greater than V occur at frequent intervals in this area. The design of bridge with pier height upto 30 m has been done by using seismic coefficient method as given in IS: 1893-1984. The values given by this method have stood the test of recent earthquake (year 2005) of 7.6 on Richter scale having epicenter near Muzzafarabad. The following additional seismic related measures have been adopted to reduce the impact of earthquake: -

a) Bridges have been mainly provided with POT-PTFE bearings and electrometric pads attached to the vertical surface of the concrete projections on top of the pier caps for seismic restraint devices.

b) Rigid structures absorb more seismic energy requiring a design for larger seismic forces than a comparatively flexible structure.

c) STAAD PRO 2002 software has been used for dynamic analysis for the idealized structure consisting of springs and member end release after a few simplifications. The longitudinal and transverse behavior has been analyzed separately so as to reduce the amount of computations and margins of errors.

5.5 Geological Investigations

Trial bore holes using heavy-duty diamond rotary core drills were carried out at each foundation location up to a depth of about 1.5 times the width of foundation below the founding level. The soil samples collected were tested for bulk density, specific gravity, uni-axial compressive strength of rock and chemical analysis. The standard penetration tests were carried out at every 30 cm depth. The founding strata consisted mostly of alternate bands of shale, sandstones, & boulder studded soil matrix.

5.6 Special Design Features of Pai Khad Bridge

The bridge at Pai Khad will have a height of 189m from bed level. This is situated in Seismic Zone V with extremely high wind speeds. The slopes at location of the bridge is of the order of 45-50 degrees on one side and vertical to sub-vertical on the other side such that the placing of any piers would have been rendered impossible, added by the existing deep gorge. Figure 5.2 shows the conceptual view of the bridge.

Figure 5.2 A conceptual view of bridge across river Pai Khad

This bridge is designed in the shape of trussed steel arch with span of 225m.The arch proposed is a three- rib arch made from large steel boxes. The chords of the trusses will be sealed steel boxes. The various special features related to the construction of these bridges are as follows: -

- Structural steel arch Railway Bridge. Material used is steel only. The structure is trussed arch.
- Bridge is designed with adequate redundancy.
- Designed for 120 years life with mechanized provisions of instrumentation.
- Various modes of vibration for the bridge are considered as it is falling in Seismic Zone V.
- In view of the extra ordinary high designed wind speeds of 220 kmph, physical topographic models of the site are tested in a wind tunnel laboratory being carried out by FORCE Technology, Norway

- Bridge is designed and checked for adequacy at various stages of construction so that partially completed structure is steady enough to resist the effect of wind/earthquake and unforeseen forces.
- Various instruments such as anemometers, accelerometers, temperature controls and central monitors are being used to monitor effects of wind and seismic loads in producing strains and loads and automatic comparisons with pre-fixed limits.
- Designed for blast loading as per IS 4991-1968.

Structural System

The bridge at Pai Khad is two-rib arch, made from large steel trusses. The chords will be sealed steel boxes, internally stiffened by filling with concrete, which will help in resisting wind induced forces. No internal access to the boxes will be available. Aesthetic merit of the bridge has been taken in consideration for a design, which will be in consonance with the nearby environment.

Codes and Design Loads

IRS standards will have the priority as regards both applicability and loads consideration. Concrete bridge code and bridge rules also shall be applicable in their area of concern. Adequate supplementary help shall be taken from UIC, BS and other international standards.

- Wind loads taken after testing carried out in the wind tunnels using models.
- Fatigue assessment of arch members done using trainload spectra specified as per BS: 5400.
- Load combinations taken as per the provisions of IRS bridge rules.

CHAPTER 6

ANALYSIS AND RESULTS

Introduction

Bridge on Pai Khad River is challenging as the gorge is 69m deep, area is having high wind speed, the soil is quite hard and seismic zone is V. The dead load, live load, trainload, braking force, wind load (with train on track and without train), seismic forces (horizontal and vertical) must be calculated as these may be subjected in lifetime of the structure. Apart from this, loads should be applied with proper consideration of structure type and properties.

6.1 Development of Conceptual Model: Two conceptual models were considered for the preliminary study:

- Two arch system, as shown in Figure 6.1
- Three arch system, main arch with two supporting arches as shown in Figure 6.2

Two arch system: For the available site, two arch system can be one possible option. To avoid constructing piers on sloping banks, long spans were necessitated; so arch bridge is suitable for this. Two arch system having equal length and center-to-center distance 12m are analyzed. Two arch system show more displacement in transverse direction. Therefore this option was not further investigated. The result summary of two arch system is given in Appendix I.

Three arch system: Three equal arches with parabolic shapes are analyzed. The arch is most suitable when uniform loading is applied and the boundary does not show any displacement. The displacements should be within safe limit. From analysis point of view it is not simple. This system makes more feasible solution for given location. This option was therefore examined further in present analysis.

Material Selection: Steel is taken as the only material for the structure due to following considerations:

- Steel can be easily fabricated and transported.
  As the structure is in seismic zone V, higher ductility can be achieved with steel, this is desirable feature for earthquake resistant design
- Steel can be easily repairable as compared to concrete.

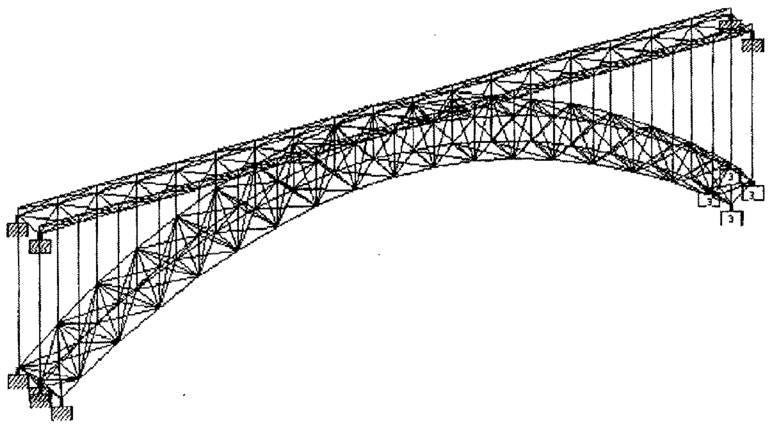

Figure 6.1 Three-dimensional model of the proposed two-rib system arch bridge

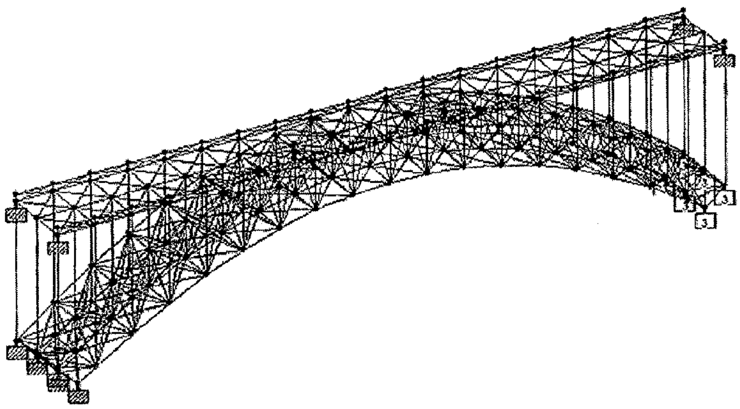

Figure 6.2 Three-dimensional model of the proposed three-rib system arch bridge

6.2 Design Criteria for Bridge: Design criteria for bridge include design philosophy, design standards, loads, material, and bearing.

Design philosophy: The following design philosophy should be adopted:

- The bridge should remain safe under design wind and design earthquake forces.
- The bridge should satisfy serviceability criteria under design combination of loading in longitudinal and transverse direction.

Design Standards:

- Wind loads taken as per IS 875 (Part 3) as per suitable zone. Wind loading should be applied on structure as per clause 2.11.2 and2.11.3 of Bridge rules 1964
- Load combinations taken as per the provisions of IRS bridge rules 1964 and steel bridge code IRS 1977 are followed.
- For seismic forces, the provision given in code IS 1893 (Part 1 – 2002) is taken.

Loads: Following primary load cases were

- **Dead load** include self-weight i.e. load of arch, suspenders, bracing.
- **Live load** for broad gauge (1.676m) is taken as sum for maximum axle load 245.2 KN for the locomotives and a trainload of 80.9 KN/m is taken. Live load due to footpath on bridge (490 KPa).
- **Tractive force** as 490.3 KN and **braking force** for axle and trainload is taken as 25% and 20% of axle and trainload respectively. For dead load and live load IRS Bridge Rules 1964 is followed
- **Racking force** for railway bridges should be 900 kg/m treated as moving load and need to be taken into account for calculating stresses of chords or flanges of main girders.
- **Wind load** for open structure is distributed as per IRS Bridge rules 1964. The bridge shall not be considered to be carrying any live load when wind is with high speed. Basic wind speed is 39m/sec for the given zone as per IS 875(Part 3). Wind load is calculated for two cases when train will be on track and when no train is on track.
- **Earthquake load** for horizontal and vertical along arch is considered as per IS 1893 (Part 1- 2002). The parameters are taken as soil type II, Damping 2%, Importance factor (I) as 2, Response reduction factor(R) as 2.5, Zone factor: 0.36, Design Earthquake is Design basis earthquake (DBE), Component of ground motion is considered as horizontal along arch +vertical ($2/3^{rd}$ of horizontal component)+transverse($2/3^{rd}$ of horizontal component). Thirty cut off mode shapes are considered.

Bearings: Bridge has been mainly provided with POT-PTFE bearings and elastomeric pads attached to the vertical surface of the projections on top of the pier caps for seismic restraint devices.

Material: Steel is taken for the whole structure.

6.3 Salient feature of proposed Bridge System:

Broad dimensions of bridge system:

- Three arch system with span 225m.
- The rise of arch is 50m and the shape is parabolic.
- The main chords of arch are box structure with cross sectional dimension 1.5m x 1.5m and are made from 25mm thick plates.
- Center to center distance between top and bottom chords= 9m.
- Center to center distance between front and back arch at the crown of bridge=20m.
- Cross sectional dimension of suspender=1m x1m
- The front, central and back arches are connected with bracing at top and bottom and at suitable intervals.

Boundary conditions:

For both arch systems end condition is fixed on one end and fixed with no moment on other end. Boundary condition on deck level is fixed on both ends.

Deflection Limit

In vertical direction	L/800
In transverse direction	L/4000

Design loads

Dead load	Dead weight of arch, deck, spandrel
Live load for single track	341.6 KN/m (Broad Gauge)
Live load of foot path of 1.5 m width	37.5KN
Impact load	63.06 KN
Braking force	73.5KN
Wind load	
Basic wind speed at 10m above ground	39m/sec
Taking effect of height and terrain condition, wind velocity at deck level	45.63m/sec
Velocity of wind at deck level for permitting the passage of train	25m/sec
Earthquake load	
Seismic zone	V
Damping	2% for steel structure

Response reduction factor	2.5
Design earthquake	Design Basis Earthquake
Component of ground motion considered	Elx+0.3Ely+0.3Elz
Material Properties	
Steel	Isotropic
Modulus of Elasticity	2.1×10^{11} N/m^2
Poisson's ratio	0.3
Mass density	7850 Kg/m^3
Arch rib	I160016A50040
Deck level	ISA70X70X8
Spandrels	LD ISA100X100X10

6.4 Analysis of Arch Bridge System

Dead load and Live load analysis: For the three arch system the dead weight of arch, spandrel deck truss are taken, in live load the train load axle load and racking forces are considered.

Wind load analysis: Wind load is taken for two cases i.e. when train is on track and when no train on track. **Train on track**: The height of train is taken as 3.5m and due to this wind on back arches is negligible as it gets shielded. **No train on track:** In this case the front arch will get maximum wind force and central and back arch will get proportionally less wind speed.

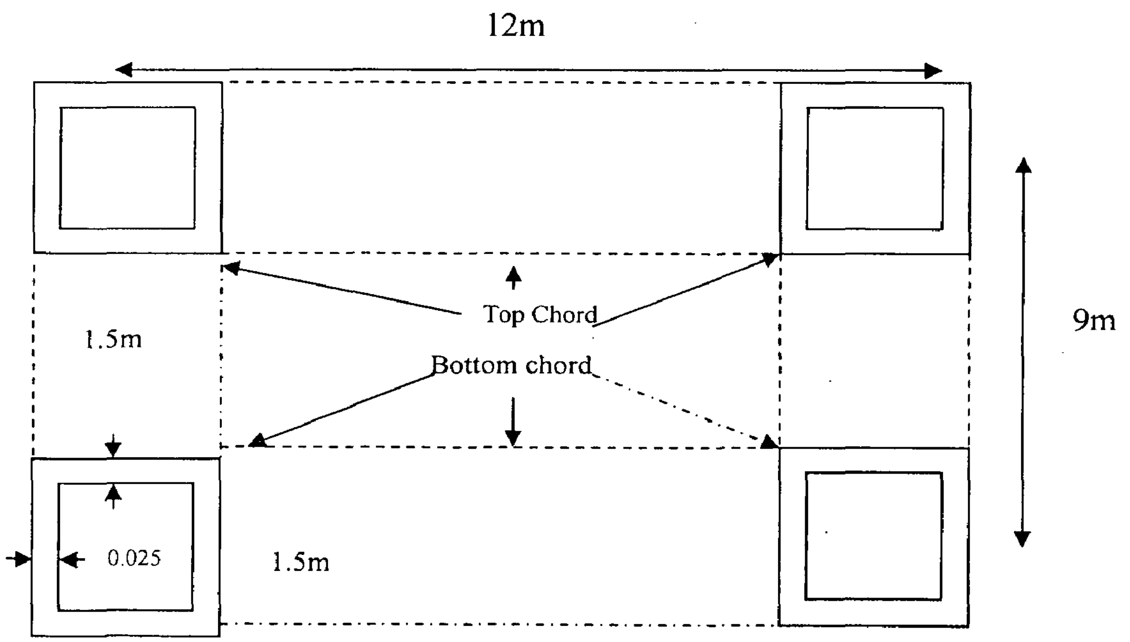

Figure 6.3 Cross section of two arch system

Earthquake load analysis: The load due to earthquake in horizontal, vertical and in transverse direction is applied. With all factors as response reduction factor, importance factor, seismic zone earthquake load can be achieved.

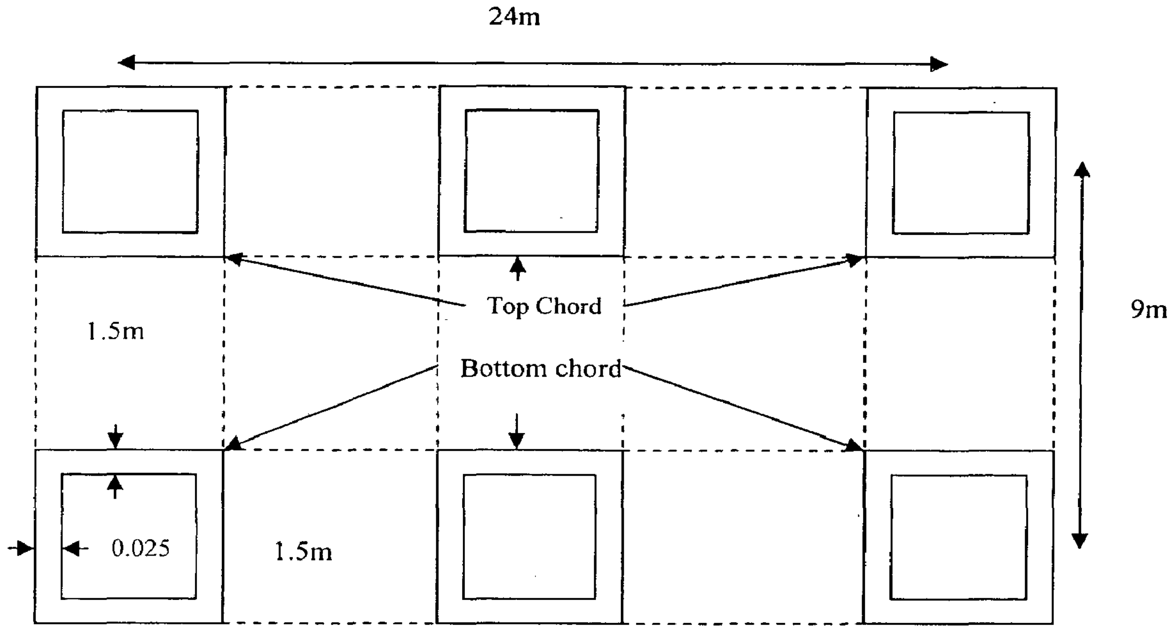

Figure 6.4 Cross section of three arch system

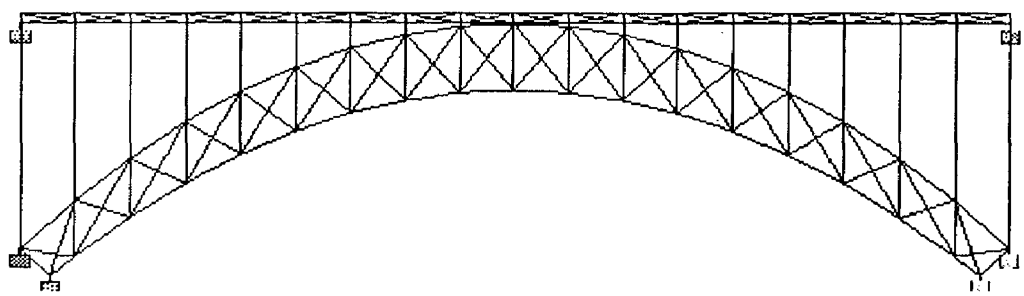

Figure 6.5 Front View of three rib arch bridge system

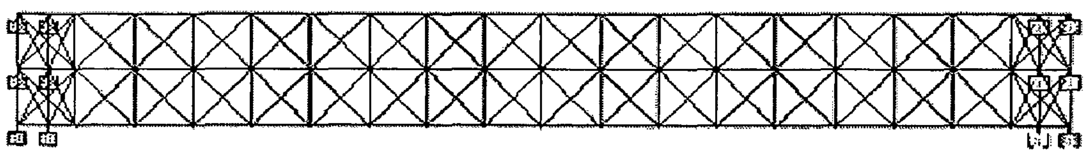

Figure 6.6 Top view of three-rib arch bridge system

Axial force: Maximum axial forces for load conditions are given in Table 6.1 and Table 6.2 for two rib arch and three rib system respectively, as only axial forces will be there due to truss system.

Table 6.1 Axial force detail for load condition of two rib arch

			Horizontal	Vertical	Transverse
Axial force	**Node**	**Load cases**	**FX** (kN)	**FY** (kN)	**FZ** (kN)
Max FX	31	DEAD LOAD+ SEISMIC X+SEISMIC Y+ SEISMIC Z	**13.8E 3**	10.5E 3	1.81E 3
Min FX	69	DEAD LOAD+ HALF SPAN LIVE LOAD	**-11.2E 3**	8.57E 3	1.08E 3
Max FY	50	DEAD LOAD+ SEISMIC X+SEISMIC Y	108.314	**11E 3**	671.317
Min FY	50	WIND LOAD WHEN NO TRAIN ON TRACK	2.04E 3	**-1.66E 3**	416.596
Max FZ	21	DEAD LOAD+ SEISMIC X+SEISMIC Y	9.38E 3	6.46E 3	**1.87E 3**
Min FZ	50	DEAD LOAD+ HALF SPAN LIVE LOAD	-11.1E 3	9.73E 3	**-1.17E 3**

Table 6.2 Axial force detail for load condition of three rib arch

			Horizontal	Vertical	Transverse
Axial force	**Node No**	**Load cases**	**FX** (kN)	**FY** (kN)	**FZ** (kN)
Max FX	31	Dead Load+Live Load+Seismic X	**19E 3**	17.2E 3	202.281
Min FX	144	Dead Load+Live Load+ Braking	**-13.9E 3**	10.7E 3	2.61E 3
Max FY	69	Dead Load+Live Load+Seismic X	-7.1E 3	**17.2E 3**	163.058
Min FY	50	Wind Load with No Train	1.36E 3	**-1.19E 3**	411.998
Max FZ	125	Dead Load+Live Load+Seismic X	19E 3	14.5E 3	**4E 3**
Min FZ	11	Dead Load+Live Load+ Braking Force	12.9E 3	11.7E 3	**-3.13E 3**

6.5 Displacements: The deflections for various load cases are shown as the result of STAAD Pro 2004 where scale as 7mm = 1m (7mm equal to 1m). Table 6.3 shows displacement for different load cases considered in analysis. Table 6.6 shows displacement at crown and at one fourth of the arch system.

Table 6.3 Maximum deflections for all load cases of three rib arch

S. No.	Load cases	Node number	Max. def. in trans Z dir (mm)	Node number	Max. def. in vertical (Y) dir (mm)
1	Dead load	124	-6.799	134	-52.803
2	Live load	124	-3.027	88	-21.891
3	Wind load when no train on track	100	-12.025	6	-2.361
4	Wind load when train on track	30	-1.377	201	-0.355
5	Seismic horizontal	161	11.490	107	43.419
6	Seismic vertical	167	2.387	10	19.962
7	Full span braking force	15	-0.202	130	-1.920
8	Half span braking force	15	-0.131	129	-1.115
9	Impact load	124	-0.559	88	-4.041
10	Braking force	15	-0.202	130	-1.920
11	Half span live load	120	-0.951	155	-17.995
12	Dead +Live	124	-9.806	134	-66.511
13	Dead+Live+Half span braking force	124	-9.823	134	-66.469
14	Dead+Wind when no train on track	124	-18.230	134	-55.086
15	Dead+Live+Wind when train on track	124	-11.180	134	-66.858
16	Dead+Live+Braking force	124	-9.807	134	-66.545
17	Dead+Live+Seismic (V)	124	-8.288	134	-47.601
18	Dead+Seismic (H)	156	9.646	107	-52.114
19	Dead+Live+Braking force+Wind when no train on track	124	**-21.259**	134	**-68.827**
20	Live +Wind when no train on track	98	-14.568	88	-21.891
21	Dead +Seismic(V)	124	-5.261	134	-33.893
22	Dead+Live+Earthquake	156	9.239	107	-65.847
23	Live +Wind when train on track	124	-4.401	88	-21.891
24	Dead+Half Span Live	124	-7.445	134	-57.954
25	Dead+wind when train on track	124	-8.153	134	-53.150

6.6 Mode shapes: The cut off mode shapes are thirty. As the seismic horizontal is dominant earthquake load so it will show mode shapes. The Figures from 6.7 to 6.14 shows first eight-mode shapes as obtained from STAAD PRO 2004 analysis for three arch rib system. All mode shapes are for seismic loading. The scale is taken as 0.01mm as 1m. Most of the nodes and beams are deflected; Table 6.6 shows that the deflections at the crown and at one fourth of the arch system for all members of bridge i.e. arch main chord, spandrel, at deck level and for guardrails for train at deck level. For earthquake loading cut off mode shape are taken at thirty. The maximum time period has been observed as 1.29 seconds, which is within permissible limit. The Table 6.4 and 6.5 gives the frequency, time period and accuracy for all thirty-mode of two rib and three rib arch system respectively.

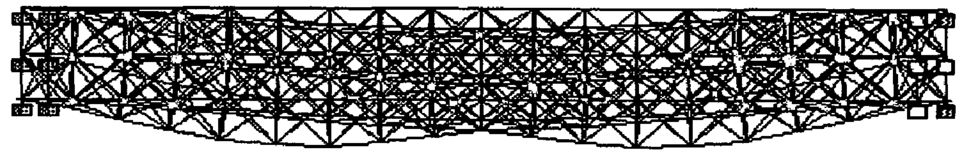

Figure 6.7 Mode Shapel

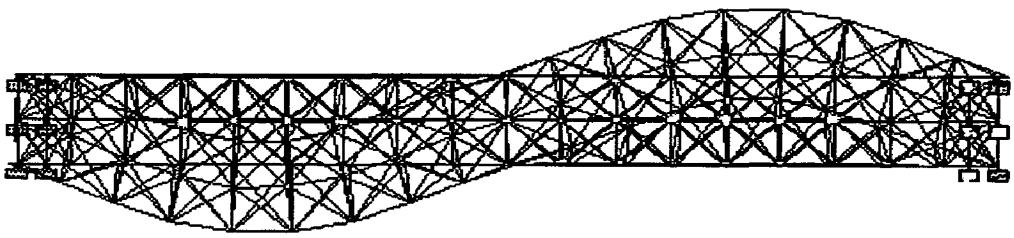

Figure 6.8 Mode Shape 2

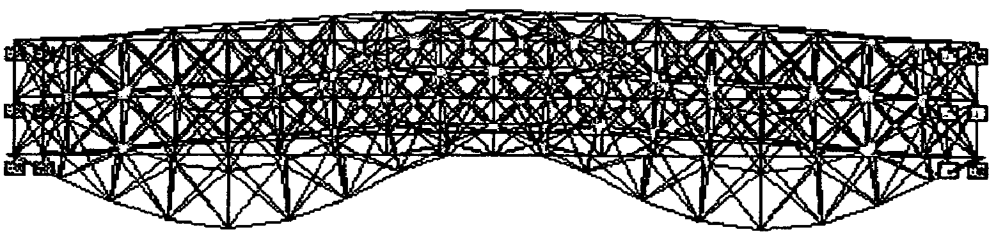

Figure 6.9 Mode Shape 3

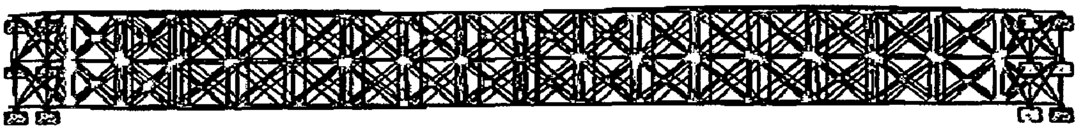

Figure 6.10 Mode Shape 4

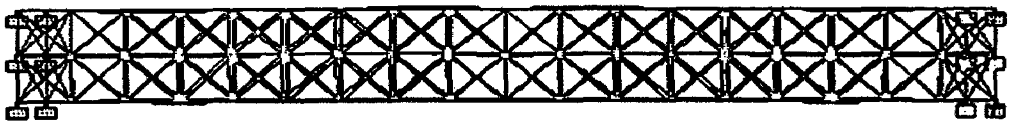

Figure 6.11 Mode Shape 5

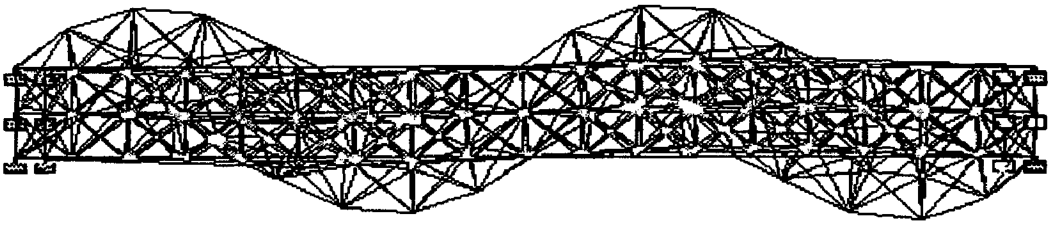

Figure 6.12 Mode Shape 6

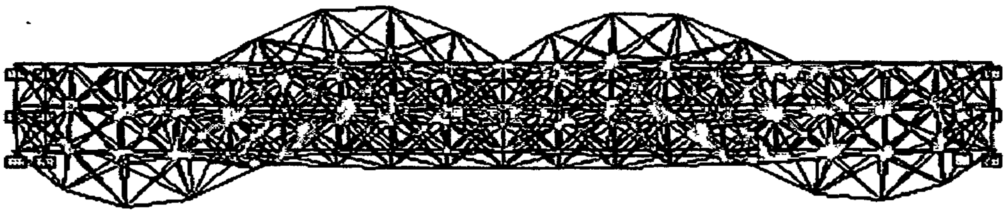

Figure 6.13 Mode Shape 7

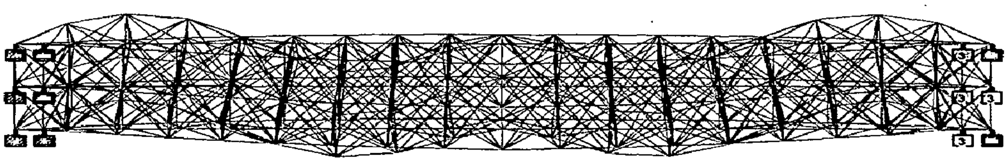

Figure 6.14 Mode Shape 8

Table 6.4 Time period, frequency and accuracy of thirty-mode of two rib arch

Mode	Frequency (cycles/sec)	Period (sec)	Accuracy
1	0.479	**2.08634**	3.917E-16
2	0.717	1.39412	0.000E+00
3	0.814	1.22845	1.358E-16
4	1.238	0.80792	1.175E-15
5	1.401	0.71383	9.171E-16
6	1.556	0.64280	0.000E+00
7	1.654	0.60451	3.946E-16
8	1.663	0.60131	9.111E-16
9	1.729	0.57828	4.815E-16
10	2.289	0.43685	8.243E-16
11	2.314	0.43210	1.344E-16
12	2.506	0.39907	4.586E-16
13	2.591	0.38599	2.145E-16
14	2.690	0.37173	0.000E+00
15	2.776	0.36029	1.682E-15
16	2.805	0.35648	3.659E-16
17	3.002	0.33312	6.391E-16
18	3.137	0.31873	4.388E-16
19	3.279	0.30501	6.698E-16
20	3.443	0.29043	7.287E-16
21	3.463	0.28880	3.603E-16
22	3.554	0.28136	5.699E-16
23	3.690	0.27102	6.346E-16
24	3.711	0.26944	4.181E-16
25	3.717	0.26902	8.337E-16
26	3.740	0.26736	0.000E+00
27	3.794	0.26357	4.001E-16
28	4.061	0.24622	8.729E-16
29	4.084	0.24484	6.905E-16
30	4.154	0.24072	1.168E-15

Table 6.5 Time period, frequency and accuracy of thirty-mode of three rib arch

Mode	Frequency (cycles/sec)	Period (sec)	Accuracy
1	0.778	**1.28472**	4.56E-16
2	0.984	1.01674	1.86E-16
3	1.14	0.87692	5.54E-16
4	1.460	0.68509	8.45E-16
5	1.788	0.55938	2.25E-16
6	2.063	0.48468	0.00E+01
7	2.098	0.47674	1.64E-16
8	2.2	0.4545	5.95E-16
9	2.237	0.44694	4.31E-16
10	2.33	0.42919	3.98E-16
11	2.418	0.41354	1.23E-16
12	2.756	0.36287	9.48E-16
13	3.005	0.33279	0.00E+01
14	3.021	0.33097	0.00E+01
15	3.1	0.32258	5.99E-16
16	3.141	0.3184	5.84E-16
17	3.239	0.30877	0.00E+01
18	3.261	0.30667	1.35E-16
19	3.314	0.30174	6.56E-16
20	3.403	0.29385	4.97E-16
21	3.461	0.28891	1.20E-16
22	3.463	0.28873	9.60E-16
23	3.48	0.28739	0.00E+01
24	3.506	0.2852	8.20E-16
25	3.597	0.278	8.90E-16
26	3.601	0.27772	6.66E-16
27	3.855	0.25942	1.94E-16
28	3.904	0.25616	0.00E+01
29	4.025	0.24848	3.56E-16
30	4.038	0.24762	3.53E-16

Table 6.6 Maximum displacements at different node of three ribs arch

S.No	Description	Node No.	X (mm)	Y (mm)	Z (mm)
1.	Crown of the arch	10	12.53	-63.981	1.713
		124	13.723	-66.704	-21.259
2.	Quarter points of arch	45	20.532	-34.960	7.394
		64	16.29	-35.902	1.541
3.	Mid point of deck	124	13.723	-68.667	-21.259
4.	Quarter point of deck	108	28.5	-36.753	6.971
5.	Spandrel at quarter point	139	21.273	-36.974	7.836
6.	Fence rail at mid point of deck	200	17.634	-68.690	2.483
7.	Fence rail at quarter point of deck	105	29.951	-43.011	5.843

CHAPTER 7

CONCLUSIONS

The purpose of this chapter is to present the conclusions based on the work that was performed during this research. The summary of whole work is presented.

7.1 Conclusion The dynamic analysis of arches shows its trivial behavior during dynamic loadings. To reduce this effect proper consideration of all dynamic factors was taken so that the structurally safe and economical bridge system could be obtained for the site under consideration. The three dimensional analysis of arch bridge for the region having seismic zone V and having very high wind speed with basic wind speed 39m/sec is a very challenging as the displacement for transverse and vertical directions is very difficult to achieve within proper limits. Arch bridge is better choice for long spans without piers. For the dynamic analysis STAAD Pro 2004 software was used. Based on the forgoing study following conclusions are drawn.

1. The two rib arch system does not hold good for the site, as the transverse displacement is higher as compared to three rib arch system. In Railway Bridge transverse displacement is an important criterion for checking of its safety. The transverse displacement in two rib arch system is 45.02 mm, which is on higher side for the railway bridge.
2. In three rib arch system maximum transverse and vertical displacements have been observed at the crown (node 124) of the arch for the load case dead + live + braking force + wind with no train on track, which are respectively 21.26 mm, and 68.83 mm. The transverse displacement for two rib arch system is 45.02 mm, which is more than double of three rib arch system.
3. The end conditions have been found to play a major role in stabilization of the bridge system. Due to this, one end was considered fixed and other end was fixed with no moments. The considered boundary condition will help in economizing the foundation system to be considered at the site. At deck level, boundary condition was considered to be fixed for both ends to achieve a structurally suitable system.
4. The site is located in seismic zone V and most severe wind zone. It was observed that earthquake load is not critical in this particular case as compared to wind.

5. As the bridge site is situated in a strategic location, three-rib arch system is comparatively more suitable to two rib arch system in case of any terrorist activism. Similar system is also being followed at the other locations of the bridges.
6. The fundamental time period for the analyzed three rib arch bridge has been found to be 1.28 seconds, which is between the permissible limits of 0.50 to 3 seconds.

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APPENDICES

APPENDIX I

Result summary for two rib arch system bridge

Displacement	Node	Load cases	X (mm)	Y (mm)	Z (mm)
Max X	108	DEAD LOAD+ SEISMIC X+SEISMIC Y	**29.546**	31.256	12.141
Min X	64	DEAD LOAD+ HALF SPAN LIVE LOAD	-17.610	-44.136	-6.997
Max Y	40	DEAD LOAD +SEISMIC Y	0.102	42.254	5.262
Min Y	40	DEAD LOAD+LIVE LOAD+ BRAKING FORCE	3.552	**-63.281**	-7.653
Max Z	5	DEAD LOAD+ SEISMIC X+SEISMIC Y+ SEISMIC Z	25.510	11.350	24.171
Min Z	97	DEAD LOAD+ WIND LOAD when no train on track	-0.272	-30.647	**-45.015**
Max rX	87	DEAD LOAD+ SEISMIC X+SEISMIC Y	19.508	-3.598	14.291
Min rX	9	DEAD LOAD+ WIND LOAD when no train on track	2.179	-30.704	-42.731
Max rY	121	DEAD LOAD+ SEISMIC X+SEISMIC Y	23.023	8.130	6.359
Min rY	130	DEAD LOAD+ WIND LOAD	-3.920	-15.307	-20.300
Max rZ	152	DEAD LOAD+ SEISMIC X+SEISMIC Y	16.854	0.101	0.133
Min rZ	115	DEAD LOAD+LIVE LOAD+ BRAKING FORCE	7.376	-0.083	0.005
Max Rst	40	DEAD LOAD+LIVE LOAD+ BRAKING FORCE	3.552	-63.281	-7.653

APPENDIX II

Mass participation factor in percent for three rib arch

Mode	X	Y	Z	Sum X	Sum Y	Sum Z
1	0	0.04	64.97	0	0.041	64.973
2	0	0	0	0	0.041	64.973
3	0	0.06	3.68	0	0.099	68.657
4	30.8	0	0	30.803	0.099	68.657
5	0	66.34	0.02	30.803	66.442	68.677
6	0.2	0	0.02	31.006	66.442	68.697
7	0	0	8.7	31.008	66.444	77.4
8	0	0.04	0.91	31.01	66.486	78.31
9	1.1	0	0	32.111	66.486	78.311
10	15.45	0	0	47.558	66.486	78.311
11	0	0.07	0	47.56	66.557	78.311
12	0	19.31	0.18	47.561	85.868	78.489
13	0	0	0	47.566	85.868	78.489
14	0	0	0.01	47.566	85.868	78.503
15	0.04	0	0	47.604	85.868	78.503
16	0	0.02	0.06	47.604	85.889	78.568
17	0	0.18	0.01	47.604	86.066	78.581
18	0	0.01	0	47.605	86.08	78.582
19	0.1	0	0	47.709	86.08	78.582
20	0.01	0	0	47.72	86.08	78.582
21	0	0.25	0	47.72	86.33	78.582
22	0	0.01	0.11	47.72	86.339	78.692
23	0	0.49	0.52	47.72	86.832	79.212
24	0.01	0	0	47.73	86.832	79.213
25	0	0	0.07	47.73	86.835	79.283
26	0	0	0	47.73	86.835	79.285
27	0	0.09	3.13	47.731	86.928	82.416
28	0.13	0	0	47.862	86.928	82.416
29	0.5	0	0	48.36	86.928	82.416
30	0	0	0.14	48.361	86.933	82.558
ZPA	11.92	0	0	100	0	0

APPENDIX III

Loading on Arch for Three rib

- Dead load: Self weight of the arches including verticals, diagonals and dead weight of fence on deck level over entire span length of 225m and dead weight of vertical suspenders.

Mass density = 7850 kg/m^3

- Live load for single track: Axle and trainload are included in live load calculation and shared equally by all three arches.

LL /node 341.63 KN

- Impact load = CDA x LL (equally shared by three arches)

CDA = 0.15 + {8/ (6+L)} =0.1846 (L=225m)

Impact load = CDA x Live Load = 63.06 KN

Impact load=63.06 KN

- Braking force for 225m span length is equal to 73.5KN to be shared by all three arches.

Braking force =73.5KN

- Racking force is taken as 8.82 KN/m as moving load.

Racking force =8.82 KN/m