SEISMIC ASSESMENT OF SIMPLY SUPPORTED Vs CONTINUOUS CONCRETE GIRDER BRIDGES
(CASE STUDY OF SELECTED BRIDGES)

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This is to certify that the thesis prepared by Yigrem Zewdu, entitled: *Seismic Assessment of Simply Supported Vs Continuous Concrete Girder Bridges (Case Study of Selected Bridges)* and submitted in partial fulfillment of the requirements for the degree of Degree of Master of Science (Structural Engineering) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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ABSTRACT

Seismic evaluations of typical concrete girder bridges are conducted for two existing simply supported bridges, which are common to the Ethiopian highway construction, and for their counterpart continuous girder bridges. These evaluations are performed for an approximate hazard level of 0.1g ground acceleration by performing linear modal response spectrum analyses on three-dimensional analytical models. The results show responses in the deck girder, reinforced concrete columns and bearings at the supports of the girders. In general, the longitudinal loading of the continuous bridges results in larger demands than the transverse loading. Consequently, the continuous nature of the deck tends to transfer seismic demand to different structural components. Therefore, the demands of piers for continuous girder bridges become larger. The simply supported bridges sustain bearing deformations in the transverse direction which are of the same value as the longitudinal response. These results suggest that both longitudinal and transverse loading are significant and should be considered when performing seismic assessment of these bridges. And also the modal response spectrum analysis result shows that the contribution of higher modes in seismic response of the continuous bridges is insignificant. Whereas it is noted that, the contributing higher modes should be considered to obtain the seismic responses of simply supported girder bridges.

CHAPTER I

INTRODUCTION

1.1 Background

Currently our country, Ethiopia, allocates much of its annual budget for the construction of infrastructures. Among these infrastructures the construction of roads require significant amount of capital and skilled man power. According to the publications of *ERA, Jan/2013,* in the previous three years, excluding loans and foreign supports, Ethiopia allocates about 35.9billion birr (about 23 % of its annual budget) for the development of road sectors. Ethiopia has spent $3.6 billion on roads over the last decade.

Consequently, construction of bridge structures, which is the major activity in the construction of roads that requires serious attention from modelling, analysis and design activity up to finalizing its construction. On the construction of roads, bridges are the major structures which are exposed to earthquake loading. Therefore, careful attentions should be employed in the modelling and assessment of bridge behaviour under such loadings.

The behaviour of a bridge structure under the influence of seismic load has been a major point of interest for engineers over a long period of time. Although significant advances have been achieved in the design and construction of earthquake resistant bridges, gaps still remain in the understanding of the seismic behaviour of bridges.

The aim of dynamic analysis is to determine the predominant bridge responses like; lateral displacements and mode shapes of bridges and to obtain overall behaviour of bridges under seismic loading. In the majority of bridge design, no dynamic analysis is required in the design process. For either situation, the dynamic analysis results contain useful information on the behaviour and condition of the overall structure. The dynamic test results do assess the current characteristics of the bridge and provide a benchmark for a comparison to subsequent evaluations of the modal parameters *(A. R. Bhuiyan et al.,* *2012).* Furthermore, dynamic analysis of bridges can be used in the assessment of bridge behaviour for seismic loading.

According to *(B. G. Nielson et al., 2005)*, in low or moderate seismicity regions, such as Hungary, there is no tradition of seismic design. There is no much knowledge of the seismic behaviour of our bridges; however it is likely that significant damage may be caused by an earthquake. Experiences on newly erected structures in the last decade show that seismic load may cause failure of some specific elements, such as foundations, piers, bearings, joints, etc. In order to achieve sufficient seismic performance, these details may have to be reinforced even though they would be safe according to static ultimate combinations.

As well referring on global seismic distribution map, Ethiopia is in low or moderate seismicity region with an approximate maximum hazard level of 0.1g ground acceleration coefficient (zone IV). In our country, *Ethiopia*, most of multi spanned bridges are being designed and constructed as simply supported, which provides a limited information of bridge seismic behaviour in the design stage and this bridge types are suggested as uneconomical in their construction industry *(BMS, 2010).*

Therefore, here in this research work an attempt is made to model and analyze an existing two multi span simply supported bridge and their counterpart as continuous T-girder bridge for earthquake loading. Then a seismic behaviour assessment was made according to their response. The appropriate geometric and bridge site data of the actual simply supported bridge is used as well in the new model input. As seen on the *fig1.1,* these bridges are selected from seismic zone (*zone-4*) of the country, by taking an approximate hazard level of 0.1g ground acceleration coefficient. This research work also places a hypothesis of, modelling and analysis of the aforementioned bridges as continuous girder provides smaller seismic demand of the super structure than that of the actual simply supported bridges. And also the study is provided with a brief study on the behaviour of bridges structure under the influence of seismic load.

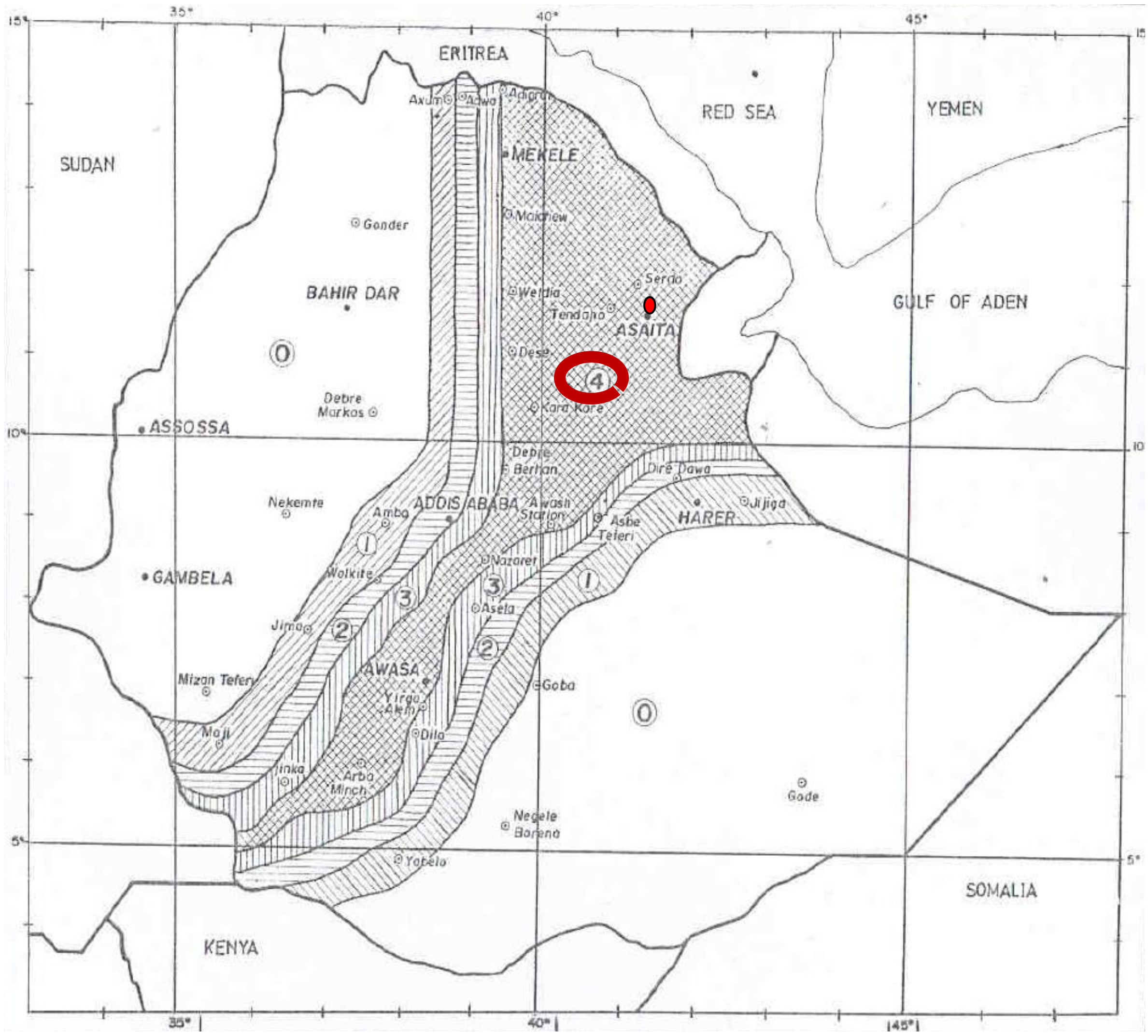


Figure 1.1 Seismic Hazard Map of Ethiopia, EBCS/1995

This study also might make important contributions to the earthquake engineering community as the effort to assess seismic hazard in the seismic zones. The generation of more reliable probabilistic vulnerability functions will be possible since a clear presentation of the inherent vulnerabilities in these bridge types is presented. This study gives support to the argument that all critical bridge components should be considered when performing a seismic assessment.

1.2 Problem Statement

In low or moderate seismicity regions, such as Ethiopia, there is no tradition of seismic assessment of bridges. Hence there is no much knowledge of the seismic behaviour of our bridges.

Currently bridge design and construction industry in Ethiopia requires a further investigation. The bridge design code, "ERA bridge design manual" which only places a detail working drawing of simply supported bridges, that only gives limited information for design and seismic assessment of bridges.

The difference in behaviour and response demand of simply supported and continuous bridges under seismic loading is another point of interest in this study. *(Dr. A. K. Sengupta et al., 2008)* presented that, bridge decks are made continuous over the supports to increase structural integrity. As well continuous beam provides an alternate load path in the case of failure at a section. A continuous beam is a statically indeterminate structure. Therefore, in regions with high seismic risk, continuous beams and frames are preferred in buildings and bridges. This study also tries to find out if multi span continuous bridges are efficient under seismic loading than multi span simply supported girder bridges.

1.3 Objectives

Generally, this study is carried out to understand the application of dynamic analysis of selected bridges. Finite element modelling of bridge structures will be conducted by using structural analysis software, SAP2000. The aim is focused on the analysis of continuous and simply supported models of bridge structures subjected to earthquake loading and study the general behaviour of the bridge types under seismic excitation.

The following tasks were performed as detail objectives of this research work;

- Finite element modelling of the simply supported and continuous girder bridges
- Perform a Modal Response Spectrum analysis
- Determine seismic demand and modal responses of the bridges
- Perform a comparative study over the seismic responses of the simply supported and the continuous bridges.

- To assess the contribution of higher mode responses

- To understand the application of dynamic analysis of bridges.

1.4 The Scope of the Study

The scope of this study is to perform dynamic analysis on two simply supported and their counterpart continuous girder bridges, located in the seismic zone IV of the country. The SAP2000V14 package program was used as a tool of this finite element analysis. The bridges' model consists of deck slab and beams system where shell elements were used in the modelling of the deck system. All data specifications and detail drawings are based on the document provided by consultant of the construction, *EZECo* and the client, *ERA*.

This research work attempts to model and employ seismic assessment of two simply supported T-girder concrete bridges and their counterpart of continuous girder bridges subjected to earthquake loading and then employ a detail study on their seismic responses. The earthquake analyses of the bridges are carried out by taking approximate hazard level of 0.1g ground acceleration. The analyses of the bridges were performed using analysis tool called Response Spectrum Analysis, which linear elastic range is recommended for such analysis tool.

The scope of this research work is not limited to model and provide the analysis results of the case study bridges, but also first by determining seismic responses and behaviour of the bridges a comparative study were provided between the simply supported and the continuous girder bridges, which the bridges have different support condition with similar aspects of other modelling parameters. The comparisons were concerned about the seismic demand and overall efficiency of the bridge structures.

CHAPTER II

LITERATURE REVIEW

2.1 Introduction

Currently, there are different monitoring techniques that have been considered to use in the structural evaluation of bridges. These include approaches based on both static and dynamic responses. The use of dynamic properties has advantages over static properties. From basics of dynamics it could be noted that, when a structure has a loading which varies with time, it is reasonable to assume its response will also vary with time. In such cases, a dynamic analysis may have to be performed which reflects both the varying load and response.

The study of vibrations requires synthesis of basic engineering science and mathematics as a special application of principles of dynamics to bodies performing some kind of repetitive motion. Vibration theory is developed by applying basic laws of nature and appropriate constitutive equations to dynamic systems. Therefore, Newton's law of motion can be used to study such vibratory motions.

Vibration can occur in many common engineering systems and if uncontrolled or not dissipated can lead to catastrophic results. Vibration of a bridge structure under the earthquake excitation is an important consideration in the design of bridges. The possibility of elastic failure of a structure with dynamic effects are neglected and in long-time repetition of dynamic stresses may lead to cumulative fatigue failures even it would be considered to be safe from static considerations. As an example, vibration of a structure induced during an earthquake lead to large stresses and can result in structural failure.

The dynamic response of a structure is dependent on a number of factors which include the main dynamic parameters of the bridge structure, the frequency content of the loads transmitted by the ground motion, the vehicles, the types of suspension systems used, the types of support condition used, the characteristic of the pavement or track irregularities, duration and the characteristic of traffic flow, etc. Significant dynamic response must be appropriately considered explicit or implicitly at the design stage.

According to *(M.J.N. Priestley et al., 1996),* seismic assessment of bridges is typically aimed at a quantification of available demand and/ or capacities of bridge sections in terms structural displacements, member forces and deformations. The flow chart that shows the process of seismic assessment of bridges is presented in *Fig. 2.1*. The seismic assessment is based on known dimensions and design details, effective section properties, and current material properties. This quantification is required for both seismic design of the new bridges and seismic assessment of the existing bridges.

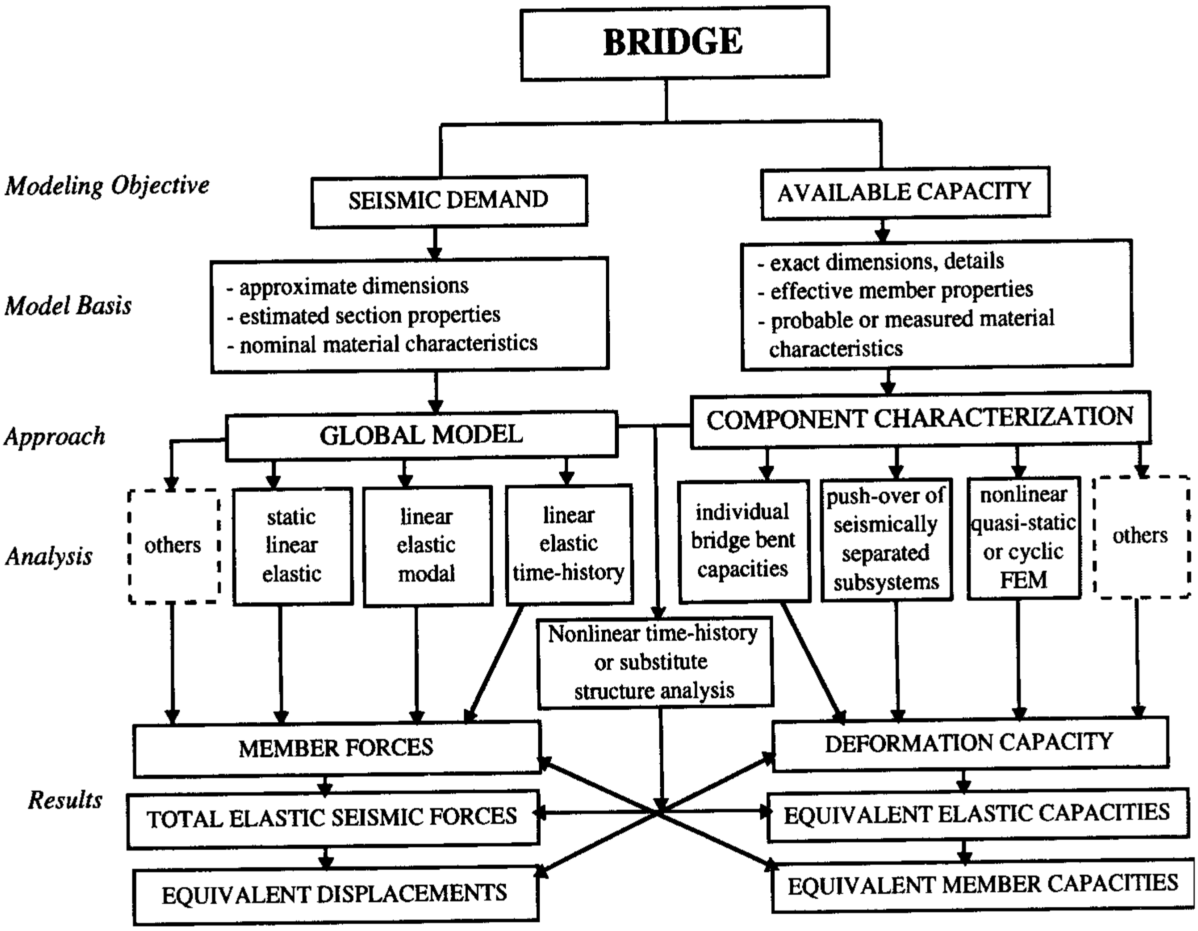


Figure 2.1 Seismic Bridge Analysis Process

Design models developed to quantify the seismic demand are often based on approximate member dimensions. To capture the seismic demand, models representing the entire structural system are developed. Various analysis techniques, mostly linear elastic, provide quantification of member forces for earthquake loading.

The seismic assessment of the case study bridges is demand analysis which the structures are expected to respond linearly. According to *(M.J.N. Priestley et al., 1996)*, linear elastic analysis is the most commonly used analysis tool for the quantification of seismic bridge responses. This analysis tool is used to predict the stiffness characteristics of columns, bents and frames, to determine deformation and force response in the linear elastic range for seismic load and to determine structural displacements.

Consequently, bridge is evaluated for ground hazard to understand typical seismic responses and highlight vulnerable components to evaluate the responses under both longitudinal and transverse excitation. The analyses in this study focus on one existing simply supported and their counterpart continuous bridge configurations subjected solely to horizontal ground motions. It is recognized that the findings from this scenario may vary as the bridge configuration number of spans, span lengths, and column height changes.

The seismic response of bridge structure is dependent on a number of factors. Among those several factors, this study investigates bridge responses for different types of support conditions used under the deck slab. *(M. Tandon, 2005)*, presented that, the common practice of placing a concrete continuity diaphragm between the concrete girders to make them continuous and thus improve vertical capacity, may often shift seismic demand from one component to another component. In particular, less deformation demand is required of the middle span bearings but more demand is required of the columns as a result. The bearings in the multi-span simply supported girder bridge appear to take the majority of the demand in the form of deformations. However, this demand appears to shift to the columns and abutments for the continuous girder bridge giving demand levels which could result in a higher level of damage to the bridge.

Thus it is helpful to realize that while attempting to improve the vertical performance of the bridge by using continuous spans one may, in fact, increase seismic vulnerability of some components. Therefore it is recommended that this phenomenon be recognized and accounted for in the design phase *(M.J.N. Priestley et al., 1996).*

Regarding deck slab continuity, *(M. Tandon, 2005)* states the behaviour of bridges that, there is a marked difference in seismic design aspects of bridges and buildings. The reduced degree of indeterminacy of bridge structures leads to reduced potential of dissipating energy and load redistribution.

In bridges, the substructures (piers and abutments) are the main structural elements which provide resistance to seismic action. For energy dissipation, ductile behaviour is necessary during flexure of these structural elements under lateral seismic loads. This essentially means that the formation of plastic hinges or flexural yielding is allowed to occur in these elements during severe shaking to bring down the lateral design forces to acceptable levels. Since yielding would lead to damage, plastic hinging is localized by design at points accessible for inspection and repair, i.e., parts of the substructure that lies from foundation upwards. No plastic hinges are, of course, allowed to occur in the foundations or in the bridge deck.

In a similar study (*Dr. A. K. Sengupta et al., 2008)* says that, bridge decks are made continuous over the supports to increase structural integrity. A continuous beam provides an alternate load path in the case of failure at a section. A continuous beam is a statically indeterminate structure. Therefore, in regions with high seismic risk, continuous beams and frames are preferred in buildings and bridges. This study also tries to find out if multi span continuous bridges are efficient under seismic loading than multi span simply supported girder bridges. Furthermore, the researcher presented the advantages of a continuous deck as compared to a simply supported deck of a bridge as follows.

- For the same span and section, vertical load capacity is more.
- Mid span deflection is less.
- The depth at a section can be less than a simply supported beam for the same span. Else, for the same depth the span can be more than a simply supported beam.
    - The continuous beam is economical in material.

- There is redundancy in load path.
    - Possibility of formation of hinges in case of an extreme event.
- Requires less number of anchorages of tendons.
- For bridges, the number of deck joints and bearings are reduced.
    - Reduced maintenance
- The higher degree of indeterminacy leads to higher ductility and energy dissipation capacity.
    - Better resistance to lateral seismic loading.

There are of course several disadvantages of a continuous beam as compared to a simply supported beam.

- Difficult analysis and design procedures.
- Difficulties in construction, especially for precast members.
- Increased frictional loss due to changes of curvature in the tendon profile.
- Increased shortening of beam, leading to lateral force on the supporting columns.
- Secondary stresses develop due to time dependent effects like creep and shrinkage, settlement of support and variation of temperature.
- The concurrence of maximum moment and shear near the supports needs proper detailing of reinforcement.
- Reversal of moments due to seismic force requires proper analysis and design.

Furthermore, regarding AASHTO *LRFD* bridge design manual, this research work attempts to model and present a case study on the seismic assessment of two existing simply supported girder bridges and their counterpart continuous bridge deck over the supports as presented in the *Fig. 2.2* and *2.3*. From the analysis result it is expected to carry out the comparison of responses for the loading in longitudinal and transversal direction of the selected bridges. In addition to that, the overall effect of continuity of bridge deck in seismic response is studied. The contribution of higher modes in the response of the case study bridges is another scenario to deal with.

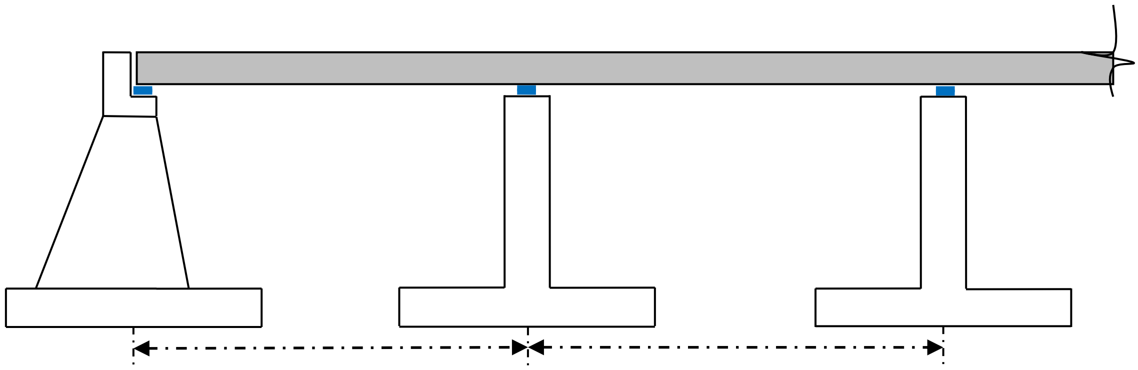


Figure 2.2 Continuous Bridge Deck

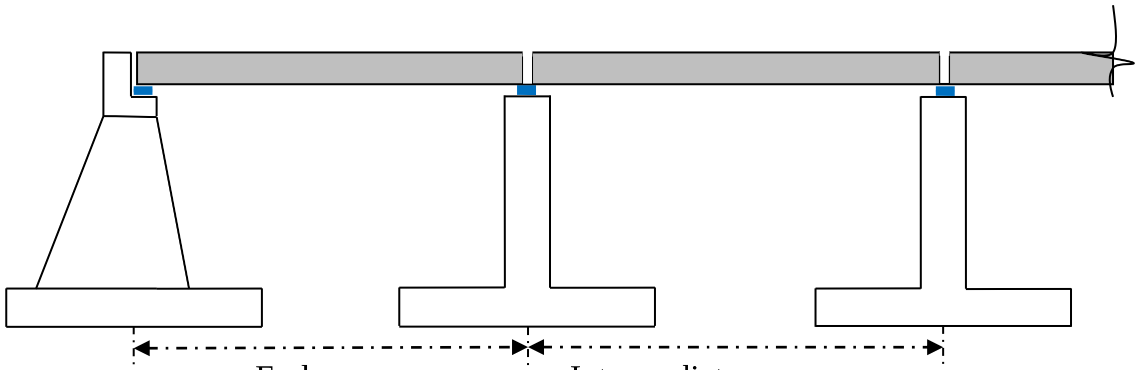


Figure 2.3 Simply Supported Bridge Deck

Generally, the seismic assessment was conducted in order to compare the seismic demand and behaviour of the bridge types under longitudinal and transversal loading. The analysis is carried out using response spectrum loading function from the design spectrum data. The study must focuses on identifying realistic analytical model that can predict and assess the actual behaviour of the bridge structures. Furthermore the difference in seismic response and behaviour of continuous deck from simply supported bridge have been studied and presented by several authors. Consequently this research work employs both of these bridge types for seismic assessment. As a result some discussions are made here after.

In another study by *(B.G. Nielson et al., 2005)*, seismic evaluations of typical concrete girder bridges were conducted for both a simply supported and continuous girder bridges common to the central and south-eastern United States. The evaluations were performed for an approximate hazard level of 2% in 50 years by performing nonlinear time history analyses on three-dimensional analytical models. The results have showed significant vulnerabilities in the reinforced concrete columns. And also longitudinal loading of the bridges results in larger demands than the transverse loading. Therefore from the results, it was suggested that both longitudinal and transverse loading were significant and should be considered when performing seismic hazard analyses of these bridges.

(K. Mahmud et al., 2010), on the research paper presented the results of a research work aimed at the analysis of a three-span concrete deck girder bridge under seismic loading. The paper used a finite element model to analyze the deck girder bridge. The research work has been carried out in two steps. In the first step, a three dimensional model of the bridge was subjected to equivalent static earthquake loading by following AASHTO code. In the second step, Response Spectrum analysis was performed. Then the design forces and moments at column bases of the bridge were obtained by using the above two methods. Then the paper found that, the design moment result found from response spectrum method is about 1.74 times of the design value obtained from equivalent static force method. Therefore based on overall findings, the research paper suggested that the response spectrum method should be performed for seismic load analysis of the bridge to achieve more reliable and safer design.

2.2 Modelling and Analysis of Bridges for Earthquake Loading

In this research work the basic objective of modelling and analysis of the bridges for seismic loading is to provide the simplest mathematical formulation of the true bridge behaviour which satisfies a particular assessment or design requirement for quantitative response determination.

Assuming that appropriate analytical tools exist to provide the numerical quantification, the model has to capture the physical and mechanical interaction of earthquake input and structure response. The objective of the analytical model is to describe the geometric property, the seismic mass, the connection and boundary conditions, and the loading of the real bridges as closely as possible to facilitate the engineering interpretation of numerical response quantities. To accomplish these, bridge superstructure and bent geometry are described in the model by relationship similar to the one given in the actual bridge structures. Individual elements simulating structural parts or complete bridge components are connected at the nodes and the nodal displacements are used as unknowns or degree of freedoms in the analytical process.

(M.J.N. Priestley et al., 1996), reported that the objective of assessment of bridges for earthquake by modelling and analysis is in general to provide some guidance to modelling and analysis of bridges for seismic loading. It is also aimed to address some of the critical issues and to review some of the fundamentals in the dynamic response of bridges. The principal objective of modelling and analysis tools is quantification of seismic response of bridges in terms structural displacements and member forces and deformations this quantification is required for both seismic design of the new bridges and seismic assessment of the existing bridges.

(N. Binti et al., 2005) presented the use of dynamic properties that has advantages over static properties in the structural evaluation of bridges, since components of the dynamic properties are only marginally influenced by variations in the loading. When dynamic properties are used, field studies have shown that it is not always sufficient to use only natural frequencies and the modal displacements. Rather some research for structural evaluation of bridges indicates that techniques based on use of derivatives of the natural frequencies and the modal displacements may be more effectively used to generate effective diagnostic parameters for structural identification. This paper presents the results of these dynamic properties for both bridges of different support conditions to compare their response and behaviour and to identify the contribution of their higher modes.

In another scenario, *(B.G. Nielson et al., 2005)* in his paper titled “Assessment of Large Engineering Structures using data collected during in-service loading” mentioned that the dynamic response monitoring has been successfully used to perform design assessments and integrity monitoring.

(A. Morris et al., 2008), also presented the experimental program clearly demonstrated the viability of full scale dynamic testing allied with FE model for assessing modal and structural parameters of a highway bridge. The application of the procedure to a relatively simple bridge showed clearly the potential for such a procedure to assist bridge managers assess their structures by providing validated structural models that may be used for load capacity assessment.

Furthermore*, (J. Simon, 2010),* reported the experimental results from ambient vibration testing of 57 typical bridges located in *California*, United State of America. Seismometers placed at the various locations on the structures and were used to measure the velocity data under normal traffic and wind loadings. The vibration frequencies and the mode shapes were determined from the measured data and compared with the finite element models, which were generally satisfactory. The author also shown that when modelling boundary conditions at the supports and section properties, care are one of an important factor.

2.3 Modal Parameters of Bridge Analysis

Regarding the past studies on the area of interest and having discussed on the general purposes of this research work, in this a detail discussion on the dynamic behaviour of bridges is presented. Bridges involve the identification of the most significant modal dynamic parameters of the structures such as natural frequencies, mode shapes and modal damping factors. The modal parameters are directly available from initial analysis of data and each will vary depending on the nature, location and severity of any damage or deterioration.

When a structure is vibrated under external force like earthquake loading, the amplitude of vibration of a damped structure may be attenuated by resisting forces developing during motion. The resisting forces dissipate energy and in time they die out the vibration. This phenomenon is known as damping. There are many causes and sources of damping, but the associated energy losses are often small and they could be neglected in the analysis of many engineering structures *(Chopra, A.K, 2007).*

It is also clear that, if the system contains some form of energy dissipation then it is said to be damped. If no external forces applied to input energy into the system then the damping will cause the amplitude of the displacements to die away with time. All real systems contain some form of damping. A common idealization used for damping is to assume that the damping force is proportional to the velocity of the structure. In this case the damping is said to be viscous and the constant of proportionality between the damping force and the velocity is the viscous damping coefficient. However, for most structural systems the damping force is small compared to either the inertia or the stiffness forces. For this reason damping is often ignored when carrying out a dynamic analysis.

With damage occurring in a structure, it would be expected to lead to an increase in damping. Unusually high damping would suggest more energy dissipation mechanisms than expected indicating the possibility of cracks in a structure. Measurement of damping in full-scale structures is often inconsistent with very poor repeatability of results. In the extraction of modal parameters it is usually the damping value which has the greatest degree of uncertainty *(M.J.N. Priestley et al., 1996)*.

If an undamped system is disturbed from its equilibrium position and no external forces are applied then it will oscillate at this natural frequency. If an undamped system is excited at its natural frequency then the amplitude of oscillations will grow linearly with time so that the response can become very large. If damping is present then the amplitude of vibration will be limited by the damping so that they will not grow to infinity.

Damping causes the peak amplitude to occur at a slightly lower frequency so that the damped resonance frequency is slightly lower than the natural frequency but for typical values of structural damping this change is so small it can be neglected. If the system is excited at some frequency other than resonance then the amplitude of the response is largely controlled by the stiffness and inertia forces *(N. Binti et al., 2005).*

There are three most common types of damping which are viscous, coulomb and structural. Viscous damping is usually associated with bodies moving through fluids at low velocities while coulomb or dry friction damping is associated with the sliding of bodies on the dry surfaces. Structural damping is concern about the internal friction of the material and it is approximately proportional to the amplitude of the displacement of the deformed body. It is independent of the frequency of vibration and it is the result of the friction between internal planes that slip and slide during deformation of the body. The energy absorbed in this manner is dissipated in the form of heat.

But in this paper the type of damping force is generated due to internal friction of the material, reinforced concrete material, hence the damping type is structural viscous damping of a damping ratio 5%. The selection of this viscous damping ratio for different types of material and structural systems can be referred from *Chopra A.K 2007* and other literatures written about dynamics of structure.

In this study once the natural frequencies of a structure have been found, the mode shapes at each of these frequencies are determined. The structures are vibrated at each of the resonance frequency and the vibration amplitude is determined for all selected points at each frequency. From the vibration amplitudes the mode shapes are developed. Normally, the mode shapes are the shapes of deformation of the structures. In general, the mode shapes of the bridge could be classified as lateral modes, vertical modes, torsional modes and longitudinal modes. The number of points required to define a mode shape accurately depends on the mode and the number of degree of freedom in the system.

CHAPTER III

METHODOLOGY

3.1 Introduction

This chapter explains more extensively on how the research is conducted. The study of the bridges' assessment for seismic loading undergoes different phases. Phases involved were very important to insure that the study meet the objective of the research.

This study is mainly involved in the modelling and analysis of two multi span simply supported girder bridges, which are located in the seismic zones of the country, and their counterpart multi span continuous girder bridges. This study is composed of several tasks in finite element modelling and modal response spectrum analysis for earthquake loading of the bridge models. The analysis is carried out using the recommended procedure as in the literature review. A three dimensions finite element model is developed with the SAP2000V14 computer package program. All the geometric data are included in the model. Starting from the deformed configuration, a modal analysis is performed to provide the vibration parameters such as lateral displacements, periods of vibrations, mode shapes and also seismic demand results of the bridges that might be used as an input in order to perform design.

3.2 Preliminary Study

A preliminary study or also known as a desk study is the first stage that involved in this work. In this stage, several references are used to obtain some information that are related in the area of interest. Among these: literatures from the previous researchers, plan and detail drawings of bridge and also the inventory of bridge. A literature review interprets and synthesizes what has been researched and published in the area of interest. This involves reading on what other people have written about the area of study. This gives the valuable information to support or refute the arguments stated and the findings of the study.

In this study, the main sources of literature review are from journals, articles and conference papers. Since the subject of dynamic assessment for bridges is quite new in Ethiopia, there are limited published documents from the local authorities. Therefore, all the articles are referred mainly from the Journals of Civil Engineering, Structural Engineering and Bridge Engineering published by different society of Civil Engineers. Besides, for the basic theory for structural dynamic and bridge engineering, the literature review is mainly from books.

3.3 Selection of Bridges for Case Study

The bridges selected in this study are located in seismic hazard zone-4 of Ethiopia. The selection of bridges is based on their exposure to seismic hazard and being common types of bridge in the construction practice. Drawings and bridge design document provide many information about the selected bridges. It gave the detail information of the bridge structures such as number of spans, bridge dimensions, type of bridges and other important data. The information obtained is used in computer analysis as an input data.

3.4 Static Analysis Verification for the Actual Bridge

This section presents the adequacy of the static design and analysis of the actual simply supported bridges. Even though this research work employs dynamic analysis, the checking of the static analysis of the bridge could be helpful to compare the manual results with the result from SAP2000 structural analysis /design program. Designs of the bridges are carried out by employing ERA's Bridge Design Manual 2002 with traffic live load of HL-93.

As stated in "*Section 2.5.2 of ERA's Bridge Design Manual 2002"*, the design vehicle live load, *HL-93* (Gross weight *325 KN*) was used in the analysis. This includes the *AASHTO* design truck *HS20-44* (gross weight *127 KN*) and lane load of *9.0 KN/m* at the same time, was found more reliable for design of bridges in Ethiopia. This design vehicle recommended based on the results of an axle load survey in Ethiopia in 1998, taking into account the frequent overloading of heavy trucks in most cases. The other live loading is the Design Tandem that represents exceptional live loading and consists of two axles each weighing *110 KN*, spaced at *1.20 m* centres. Furthermore, all weights of the structure components including the overlying wearing course are taken as dead load. The load combination to be used for design is Strength - I Limit State, *1.25 x DL+1.75 x (LL+IM),*

Up on the completion of the static analysis using the program the results are checked accordingly with the manual calculation. For the purpose of illustration the entire span moment envelopes from the actual bridge design and from *SAP2000* design software are presented in the *Figure 3.1* and *Figure 3.2*, respectively. Then having the results of the actual bridge, the analysis results are compared in order to find out if the program could be utilized to carry out response spectrum analysis of the case study bridges.

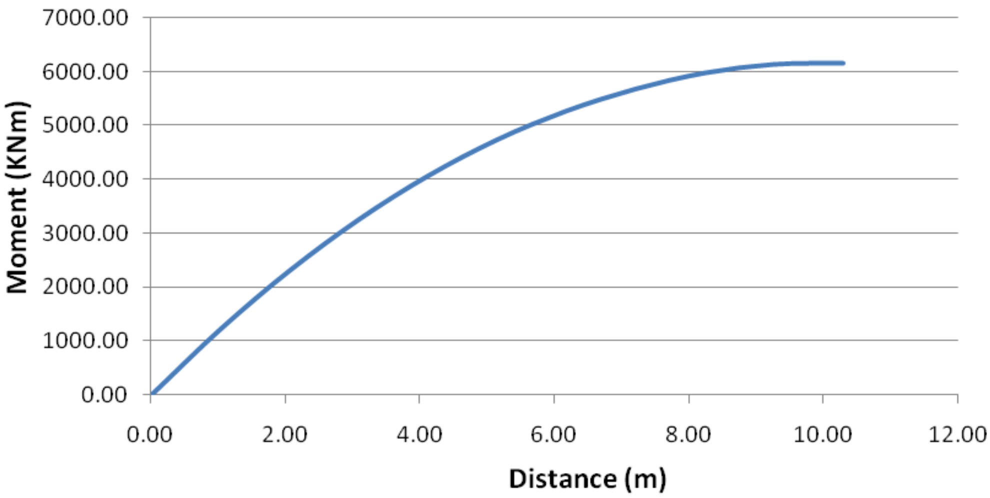


Figure 3.1 Moment envelope of the Bridge from Actual Design

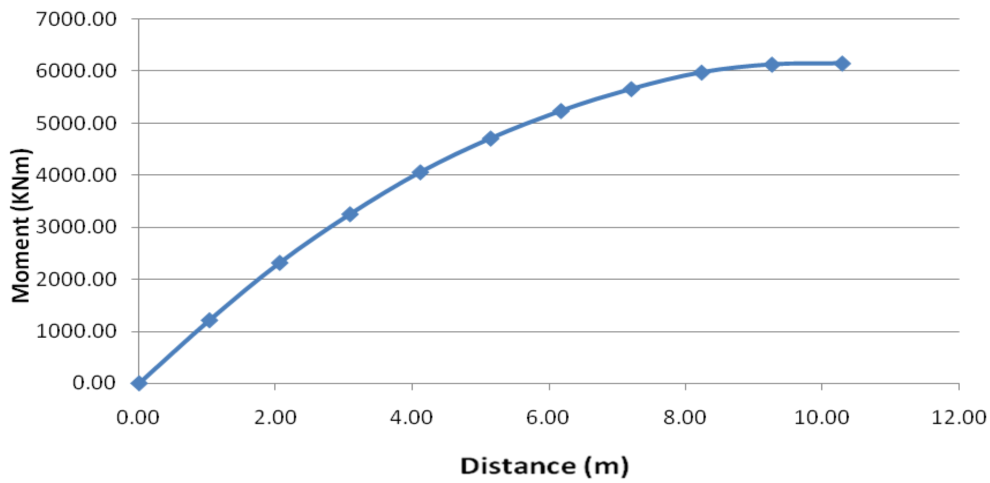


Figure 3.2 Moment envelope of the Bridge from the Analysis Program

The static analysis result of the bridge is presented in the figure above. Hence one could observe that the maximum mid-span moment envelope of the simply supported bridge from the actual bridge design result as well as from the analysis program is presented. Consequently, it is found that the maximum mid-span moment of bridge for both analysis conditions results similar values, giving about *6155 KN-m*. Therefore, it is appropriate to employ the bridges in the case study as well as it adequate to utilize the program as analysis software.

3.5 Finite Element Modelling and Modal Analysis

In finite element modelling structure will be divided in to elements and very elements will be descritized in to a simple geometric shape called finite elements. Other parameters such as material properties and geometry are also considered and expressed in terms of unknown value at element corner *(A. Morris et al., 2008).*

In this study finite element models for both bridge support conditions were subsequently developed. In these models, the beam was modelled as three dimensional beam element and the deck slab was modelled as three-dimensional shell elements in one single nodal lines which the slab and girders are monolithic as obtained from the detail drawing.

Then modal response spectrum analysis is needed to determine the basic modal properties for the subject bridges; periods, maximum amplitudes, natural frequencies, and mode shapes and also to obtain stresses, moments, forces and the capacity demand ratios of the multi span simply supported and multi span continuous girder bridges subjected to seismic loading. Having the seismic responses and force demands for the multi span simply supported and multi span continuous girder bridges, comparisons were made between the subject bridges.

Generally the methodology of the research activities from desk study and bridge model development to data analysis and discussion involved in this study are summarized and presented in six phases as shown in *Figure 3.2.*

<u>Discussion, Reading and Observation</u>

- Find the problem through reading, discussion and observation of the interest area.
- Identify the scope of project, aim and objectives of the project.
- Understand and identify the background of the problem.
- Study the feasibility and the needs to carry out in the modeling.
- Plan the methodology for the study

<u>Preliminary / Desk Study</u>

- Understand the background of the problem.
- Understand the basic theory for structural dynamic.
- Identify the dynamic properties that will change subjected to EQ
- Identify the method of analysis to obtain the dynamic properties.
- Identify the finite element software that is sufficient in this study.

<u>Selection of Bridges for Case Study</u>

- Identify the suitable bridge for case study.
- Collect the bridge data information as an input data in modeling.

<u>Data Analysis</u>

- Analysis using Modal Response Spectrum Analysis
- Determine dynamic properties and Seismic demand from the analysis.

<u>Discussion, Conclusion and Recommendation</u>

- Discuss the results of simply supported and continuous case.
- Discuss the modeling issue involve this study.
- Draw an overall comparison and conclusion for this study.
- Give some recommendations for future research or project.

Figure 3.3 Summary of Methodology for the Study

CHAPTER IV

MODELLING AND ANALYSIS OF THECASE STUDY BRRIDGES

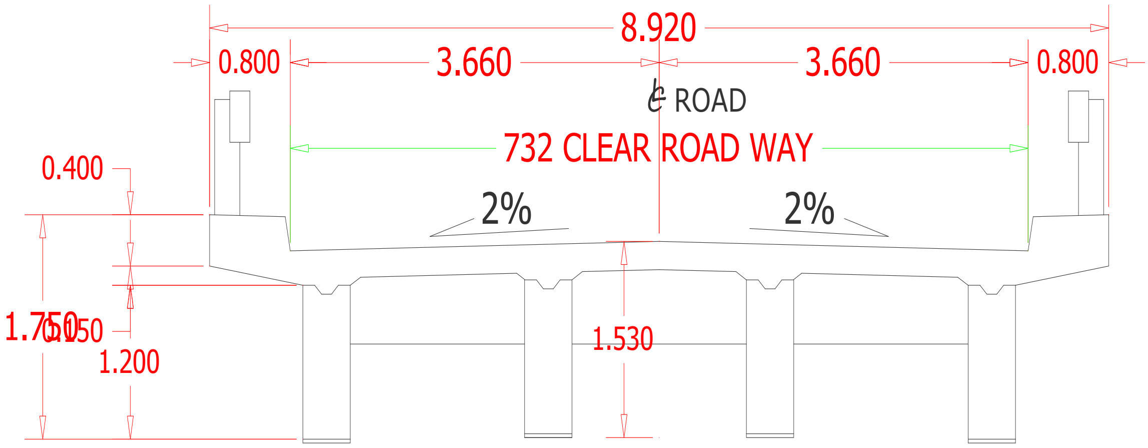
4.1 Introduction

Now days engineers use computer software for design and analysis of structures. There are lots of engineering soft wares that can be used in civil engineering field. Using computers engineers can design and analyze the structure with less error and time particularly for complex structures such as bridge decks. Bridge decks can be analyzed through a variety of models in 2D and 3D.

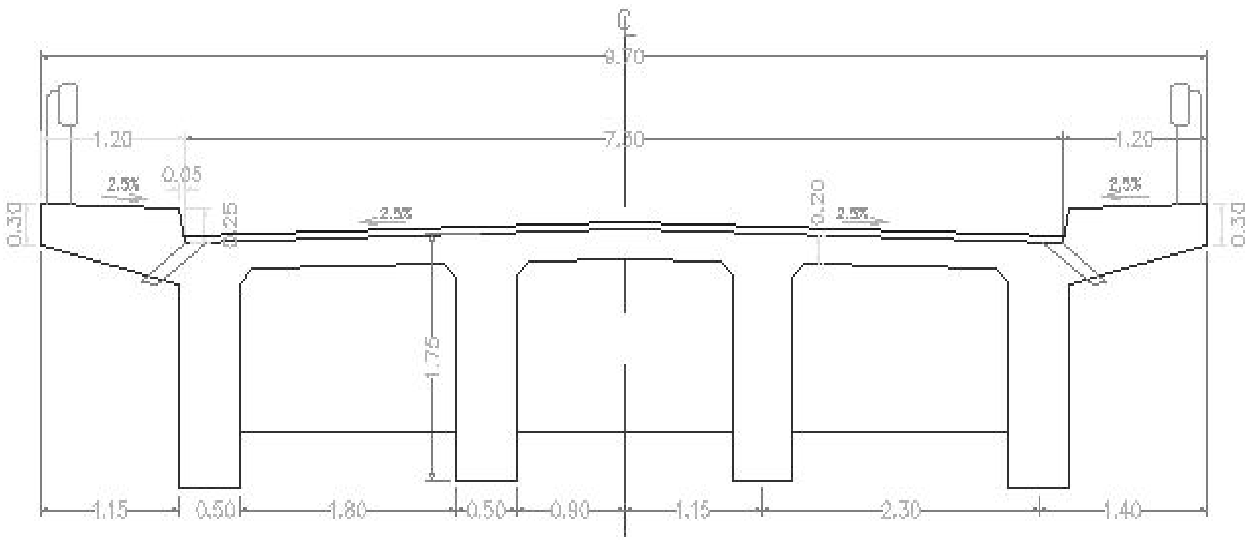
According to (*M.J.N. Priestley et al., 1996),* using programs there are several basic methods for developing models of a bridge superstructure consisting of concrete slab deck on a series of concrete girders. One method involves the use of two layers of nodes. The first layer consists of plate elements to represent the deck slab and the second layer consists of beam element to represent the beams. A second method represents the entire bridge in one layer of nodes by treating the beams and deck as composite beams. A third method treats the beams and deck as separate elements (beam elements and shell elements) but in the same nodal layer.

In general, all of those models give reasonable solutions for the first mode of vibration but differ significantly in their ability to predict subsequent modes. In this study, the second method was selected to model the two bridges. The model consists of the entire bridge in one layer of nodes by treating the beams and deck as composite beams. The beam elements are used to represent the concrete diafragms and the shell elements are used to represent the cast in-situ concrete deck girder. Both element types have same degree of freedoms, six degree of freedoms and nodes are coinciding with each other.

But the bridges under this case study employ the second method of modelling, where the girders are worked in such a way that the in-situ concrete is casted monolithically with the slab. The case study bridges which are selected from *Afar Regional State* road project are presented in Figure *4.1* and *Figure 4.2* that show the longitudinal and cross sectional views of the existing bridges.

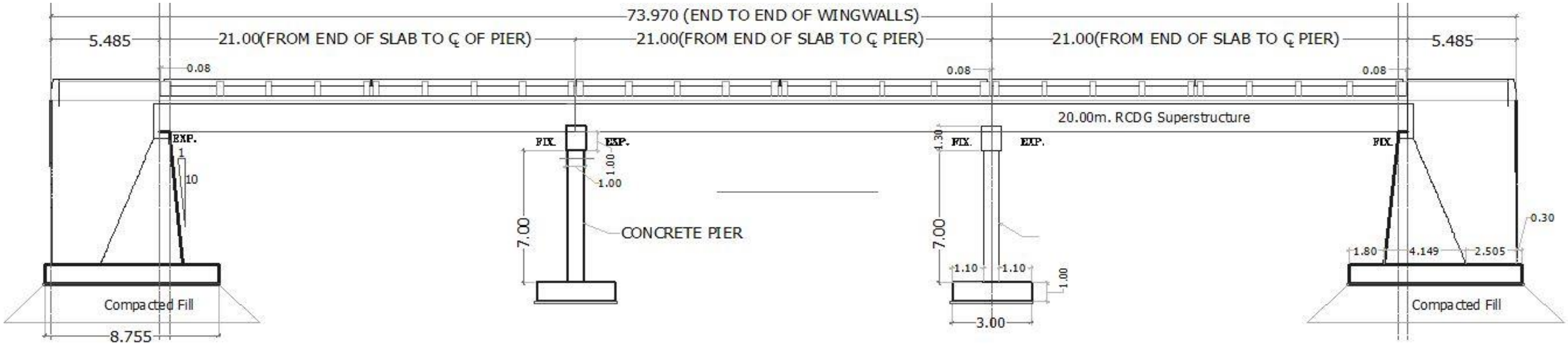


(a) Bridge 1

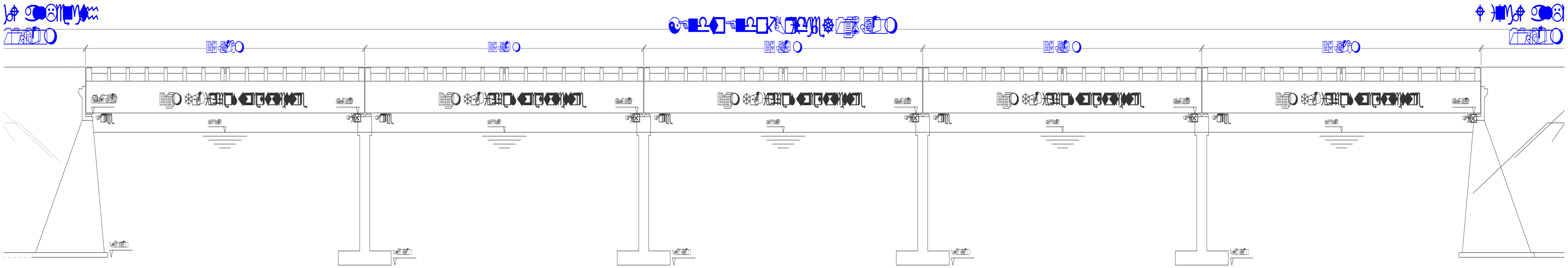


(b) Bridge-2

Figure 4.1 Cross-sectional Drawing of (a) Existing Bridge-1, (b) Existing Bridge-2



(a) Bridge 1



(b) Bridge 2

Figure 4.2 Longitudinal-section View of Existing Bridges (a) Bridge 1 and (b) Bridge 2

4.2 Finite Element Modelling

Finite element analysis is an effective method of determining the dynamic performance of structures for three reasons which are saving in design time, cost effective in construction and increase the safety of the structure. A finite element analysis is a numerical simulation of the behaviour of a real-world structure which is intended to provide information that can be used by a designer or design team to ensure a structural design is fit for purpose when it enters in-service operation.

The process of setting up an analysis requires that such factors as the loads applied to the structure, the structural behaviour and responses, the boundary conditions, etc., are all represented by a set of mathematical functions or operations. This is an important concept to understand because the focus of the analysis is the real world, which is not a mathematical model *(K. Mahmud et al., 2010).*

This research work presents a finite element method of analysis which basically consists of dividing the bridge decks and beam elements into a number of parts, called elements. The modelling of the bridges consists of key points/nodes, lines, areas and volumes with increasing complexity in that order. These are connected to each other at their nodes as shown in *Figure 4.3*. Each node may have six degree of freedoms which are defined as independent displacements (three translation and three rotations) used to describe the movement of each node. The deck is modelled using shell element which has maximum side dimension of one meter and the type of elements should be selected so that the deformed shaped of the structure can be adequately represented.

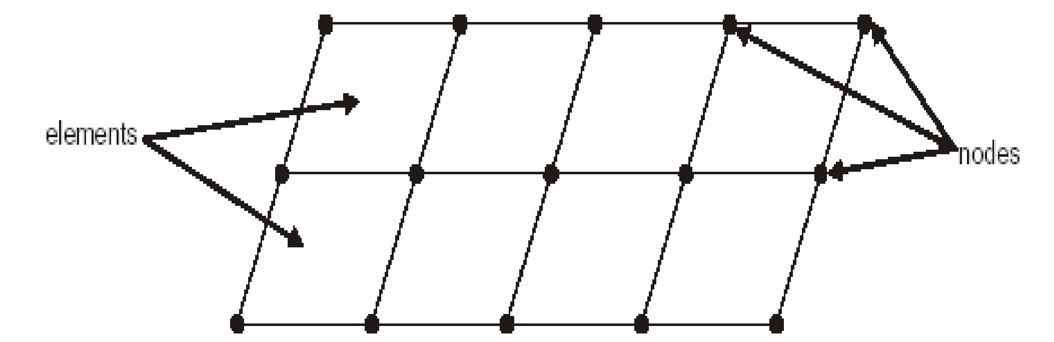


Figure 4.3: Elements and Nodes in Finite Element Analysis

(A. Morris et al., 2008), also generalized the features of finite element analysis as: There are many reasons why the evaluation of a structural analysis and design might require the use of several finite element models.

- Convert the real world system into a mathematical description.
- Turn this description into a form which allows a computer to be brought into the picture to solve this mathematical problem.
- Take this output and turn it back into parameters that relate to the real-world structural behaviour.

The bridge system is modelled using finite element method using structural analysis software, SAP2000v14. The model is based on approximate member dimensions from a preliminary design, utilizing estimated effective section properties and design material characteristics to capture the seismic demand. The software incorporates bridge design wizard in order to input the bridge properties and interactive graphical facilities. Regarding AASHTO LRFD bridge design manual, the analysis package performs dynamic analysis for earthquake loading. Upon the completion of analysis of the bridges program could generate seismic design report as a readily assimilated form.

The choice of element types is an important criterion in finite element modelling of the bridges. The choice of element type depends upon the problem that is to be solved and what is required from the analysis. If it is only necessary to find the resonant frequencies then a relatively coarse mesh of simple elements is required. Thin walled structures such as beams and slabs are always prone to vibration with relatively low natural frequencies. In such cases the structure can be modelled using beam and shell elements.

These models based upon the centre line geometry of the structure, means that the precise details of connections are not well defined. Provided that the connection is relatively stiff then this detail tends not to affect the low frequency modes and it is not necessary to include the detail. However, there are some connection details that must be included. If plates are reinforced by stiffeners they must be included in the model. If the plate is modelled as shell elements and the stiffener section is modelled as beam elements. In this study Beam and shell elements were utilized in the modelling of the bridge system. Many aspects were considered in choosing the element types such as:

• What elements are available in the system?

• Which elements have past experience shown to be accurate for the type of problem under consideration?

• What elements are recommended for use?

As with a static analysis, if there is any uncertainty about the form of response or the best element types to use, one or more simple preliminary studies should be undertaken before the full analysis is carried out. These consist of very simplified models of the problem or some parts of the problem that allow the user to gain understanding of what the results will be and to access the effects of the change of any parameter defining the structure. Such studies are extremely useful in dynamics since it can be difficult to use intuition to say what the response will be like.

After selection of the appropriate element type, there are lot of methods in constructing beam and slab bridges, the most known is an in-situ concrete slab on steel or precast concrete beams, or steel beams with a composite steel or concrete slab or even an entire of in-situ beam and slab. In this study the construction of the girders were usually worked in such a way that in-situ concrete is casted monolithically with the deck slab.

Beam element is used to model plane and space frame structures. A variety of thin and thick beams in three dimensions are available. Beam element may be straight or curved and may model axial force, bending and tensional behaviour. In this study beam is a three dimensional straight beam element and include of shear deformations. Along the member of the beam, the geometric properties are remaining constant. The number of nodes is defined with end release conditions. The beam element is defined by two nodes, the cross-sectional area, two area moments of inertia (I_{ZZ} and I_{YY}), two thicknesses (t_y and t_z), an angle of orientation (θ) about the element x-axis, the torsional moment of inertia (I_{XX}) and the material properties. The typical section properties of the bridge structural elements are tabulated in *Table 4.1.*

Table 4.1 Geometric Properties of the Case Study Bridges

Description		**Bridge-1**		**Bridge-2**	
		Continuously S.	Simply-Sup	Continuously S.	Simply-Sup
Span	*L (m)*	21.00	21.00	25.25	25.25
No of Span	*no*	3	3	5	5
Slab Thickness	*t (mm)*	200.00	200.00	200.00	200.00
Total Depth	*D (mm)*	1350.00	1500.00	1650.00	1750.00
Total Width	*w (mm)*	8920.00	8920.00	9700.00	9700.00
Roadway Width	W_1 *(m)*	7.30	7.30	7.30	7.30
Sidewalk Width	W_2 *(mm)*	800.00	800.00	1200.00	1200.00
Web Width	b_w *(mm)*	470.00	470.00	500.00	500.00
Fillet	*f (mm)*	100.00	100.00	100.00	100.00
Girder clear spacing	*S (mm)*	1730.00	1730.00	1800.00	1800.00
No of Girders	*no*	4	4	4	4
No of Cross-Diaphragms	*no*	4	4	4	4
Diaphragm Depth	d_d *(mm)*	1000.00	1000.00	1000.00	1000.00
Diaphragm Width	b_d *(mm)*	300.00	300.00	300.00	300.00
No of Column/Bent	*no*	2	2	2	2
Height of Column	*h (mm)*	7.50	7.50	7.98	7.98
Pier cap Depth	*(mm)*	1000.00	1000.00	1200.00	1200.00
Pier cap Width	*(mm)*	1000.00	1000.00	1250.00	1250.00
Circular Pier diameter	*(mm)*	800.00	800.00	900.00	900.00
Girder Sectional Area	$A\ (mm^2)$	575000.00	650000.00	725000.00	775000.00
Weight of deck-girder	*W (KN/m)*	103.800	111.360	107.88	127.08
Neutral Axis	$Y_t\ (mm)$	475.00	550.00	625.00	675.00
	$Y_b\ (mm)$	675.00	750.00	825.00	875.00
Moment of Inertia	$I_{XX}(mm^4)$	$6.34\text{x}10^{10}$	$9.14\text{x}10^{10}$	$12.7\text{x}10^{10}$	$15.51\text{x}10^{10}$
	$I_{YY}\ (mm^4)$	$1.24\text{x}10^{10}$	$1.35\text{x}10^{10}$	$1.51\text{x}10^{10}$	$1.61\text{x}10^{10}$
Section Modulus	$Z_t\ (mm^3)$	$1.88\text{x}10^7$	$2.52\text{x}10^7$	$3.25\text{x}10^7$	$3.80\text{x}10^7$
	$Z_b\ (mm^3)$	$3.79\ \text{x}10^7$	$4.69\text{x}10^7$	$5.67\text{x}10^7$	$6.38\text{x}10^7$

Source; EZECo. And ERA

Beam elements were chosen to represent the diaphragm as cross bracing and the pier cap beam. Diaphragms are not a factor in symmetric response such as longitudinal bending modes but may have a significant influence on transverse bending modes and torsional modes.

The other element type employed in the modelling is shell element. Shell element is also developed assuming that the thickness of the component is small relative to the other two dimensions and is also modelled by their middle surface. They differ from plate elements in that they are considered to have six degree of freedom at each node, three translations and three rotations.

Shell has both bending and membrane capabilities. Both in-plane and normal loads are permitted. The element has six degrees of freedom at each node which is translation in the nodal x, y and z directions and rotations about the nodal x, y and z-axes. Stress stiffening and large deflection capabilities are included. A consistent tangent stiffness matrix option is available for use in large deflection. This was chosen for utilisation in the modelling of the cast-in-situ concrete deck-slab.

The shell element is defined by four nodes and four thicknesses. In this study, the thicknesses are assumed constant in all nodes. Both bridges have 200mm thickness for concrete deck slab different depth of girders depending on span length. The 10mm thickness of asphaltic concrete wearing course was not included in this modelling as an added mass. The typical cross sectional dimensions of the bridge's superstructure are presented in the *Fig. 4.4*, *Fig. 4.4* and *Fig. 4.6*.

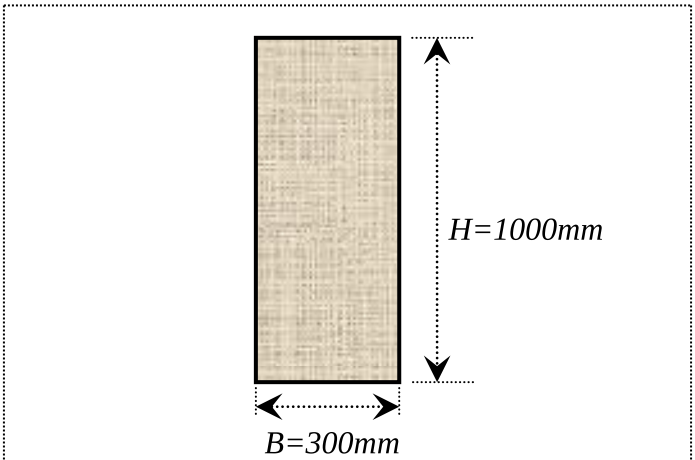


Figure 4.4 Typical cross sectional dimensions of cast in situ cross diaphragm

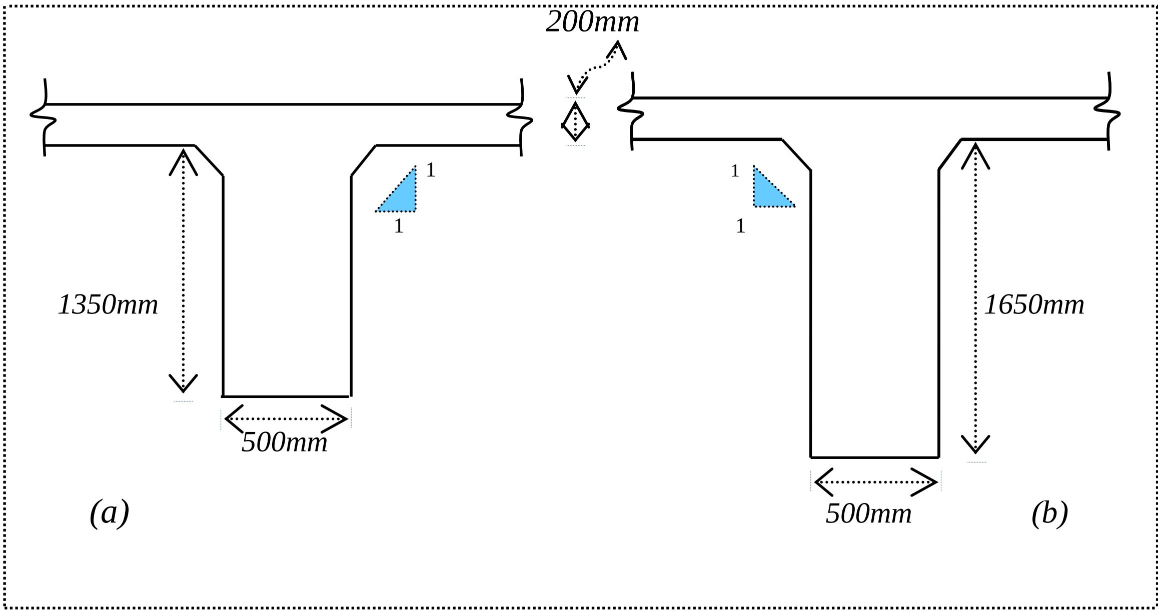


Figure 4.5 Dimensions of Cast in situ Simply-Supported Concrete Girder for (a) Bridge-1 and (b) Bridge-2

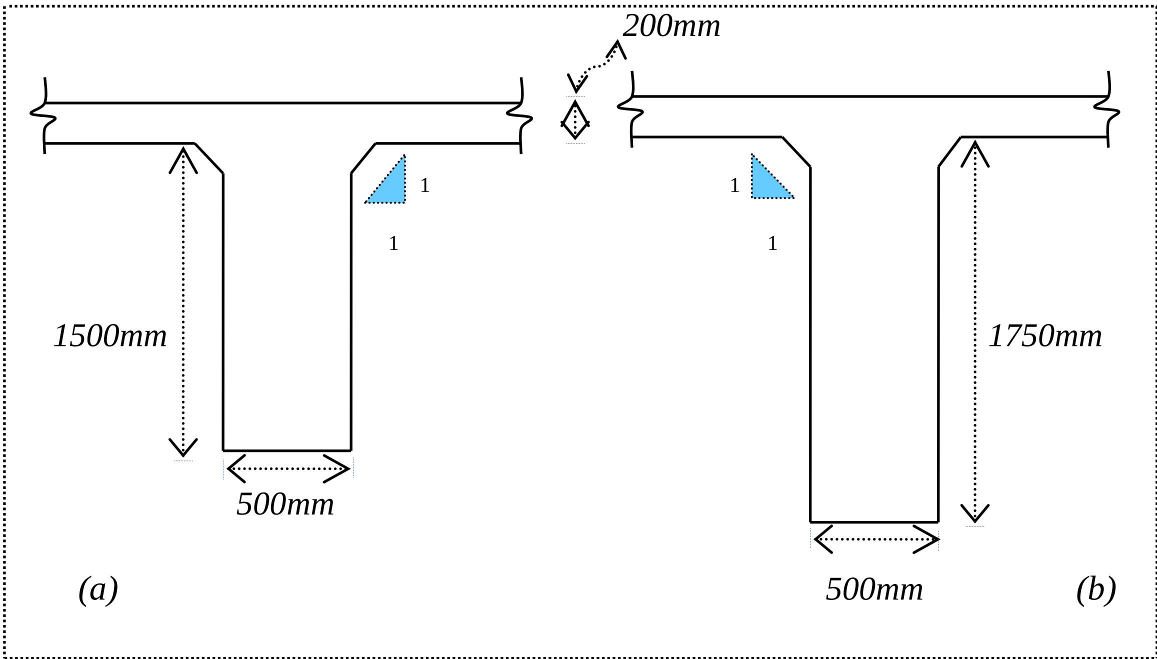


Figure 4.6 Dimensions of Cast in situ Continuous Concrete Girder for (a) Bridge-1 and (b) Bridge-2

4.2.1 Material Properties

Similar material properties are used for structural elements such as monolithic deck-girder and piers. The structural material used in this research, cast in situ concrete which has a grade of C-30 concrete for major structural components and C-10 as lean concrete at the foundation. The most important input data in dynamic analysis is to consider the mass density of structure. It gave the actual dead weight of the model structures. The mass density for the materials and other material properties used in the model are obtained from the design firm, EZECo , that comply with "ERA's Standard Technical Specifications 2002" and/or "AASHTO LRFD Bridge Design Specifications 2005 editions".

Table 4.1 and *Table 4.2* shows the material and dynamic properties used in the modelling and analysis of the bridges, respectively. The basic material properties of the bridges are taken from the actual detail drawing.

Table 4.2 Material Properties for Case Study Bridges

Concrete						
Class of concrete	MinCube (150mm) compressive Strength, f'c (MPa)	Min Cylinder compressive Strength, f'c (MPa)	Poison's Ratio (v)	Young's Modulus Ec, N/mm^2	Mass Density (Kg/m^3)	Unit Weight, KN/m^3
C-30/20 for structural components	30	24	0.2	24768x10^6	2400	24
Cement mortared stone masonry works shall comply with the requirements of material properties stated in section 8904 of ERA's Standard Technical Specifications 2002.						

Reinforcement Steel					
Diameter of bar	Grade of Steel	Min Yield Strength Fy (MPa)	Young's Modulus, Es (N/mm^2)	Unit Weight (KN/m^3)	Modular Ratio (n=Es/Ec)
All Diameters	400	400	200x10^9	78.5	9.07
Bearing	Steel Bearing				

Table 4.3 Dynamic inputs of the bridges

Seismic properties of the Bridges		
Description	Bridge-1	Bridge-2
Design code	EBCS/AASHTO-LRFD	
Seismic Zone	IV	
Acceleration Coefficient(α)	0.1g	
Site Classification	III/C	
Damping ratio	5%	
Analysis type	Modal Response Spectrum	
No of modes carried out	18	30

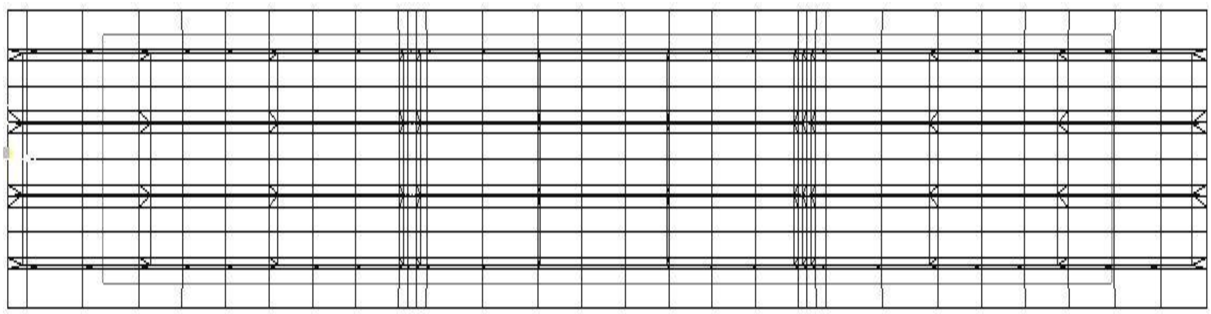
4.2.2 Loading and Boundary Conditions

Several other attributes were added in an attempt to increase the reliability and adaptability of the model. This could be achieved through providing an appropriate finite element discretization of structural elements and taking care in the assignment of boundary conditions.

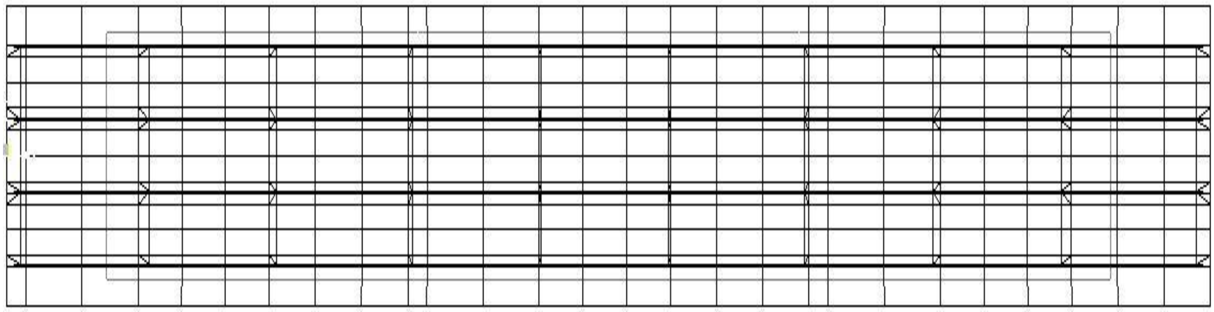
In constructing a finite element mesh for a dynamic analysis there are a range of aspects of the problem that have to be considered. The stiffness distribution is important in static analysis whereas for dynamic analysis, the mass distribution is the most important. The requirements of the result of the analysis can be different for static and dynamic analyses.

Very often the purpose of the static analysis is to find peak stress values associated with stress concentrations. For a dynamic analysis the stress distribution is often less important than knowing the resonant frequencies or the displacements associated with the dynamics response. Therefore, in the case of dynamic response the more regular mesh are required in order to get the resonant frequencies.

In this study, the models are discretized in to a minimum of one meter edge length. The numbers of element divisions are 21 for Bridge 1 and 25 for Bridge 2 along each span length. *Figure 4.6* shows the finite element discretisation for the bridge models.



(a) FEM Discritazation of the Simply supported Bridge



(b) FEM Discritazation the Continuous Bridge

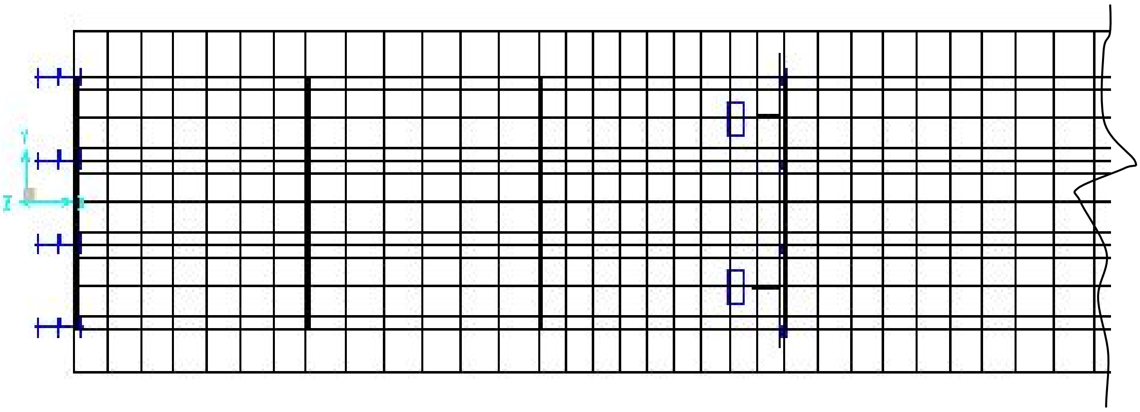
Figure 4.7 Finite Element Discretisation of Case Study Bridge-1

One of the major problems associated with a dynamic analysis is the modelling of boundary condition. These boundary conditions are the limitations on movement of the structure at places such as anchor locations. They will have mass and stiffness and can respond dynamically. Generally a high stiffness is associated with a relatively high mass so that the natural frequencies of the supports can be of the same order as some of the higher important structural frequencies.

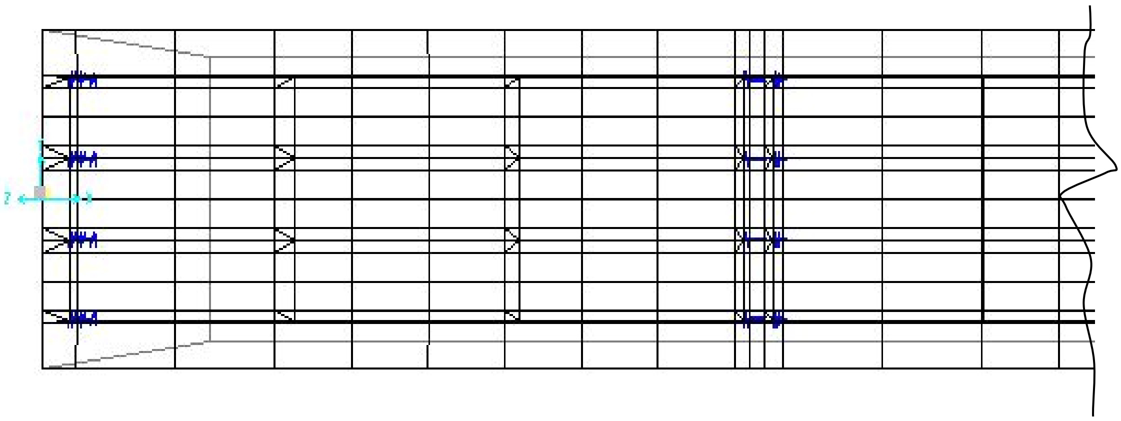
For the case of continuous deck model both bridges are modelled as having pinned support assigned at all interior supports of the beam and roller support at abutment ends. Whereas for the simply supported case both bridges are modelled as having pinned support assigned at one end of the beam whereas the other end is specified as roller support. The pinned support is constrained against translation at x, y and z directions whereas the roller support is constrained against translation of z and rotation about longitudinal directions based on global coordinate system.

The mechanical loads are probably the most common forms of loading. The finite element method only recognizes loads applied at the nodal points and the raw form of loading data consists of specifying a set of nodal loads. The mechanical loadings involved in this modelling are pressure loading and self weight loading.

In addition to the dead load, a supper dead load pattern is created in order to represent the parapets and the wearing surface load above the deck slab. For the purpose of comparison in mid span moment between simply supported and continuously girder bridges, a vehicle live load with appropriate load combination is used. *Figure 4.7* shows the boundary conditions and loadings for bridge model.



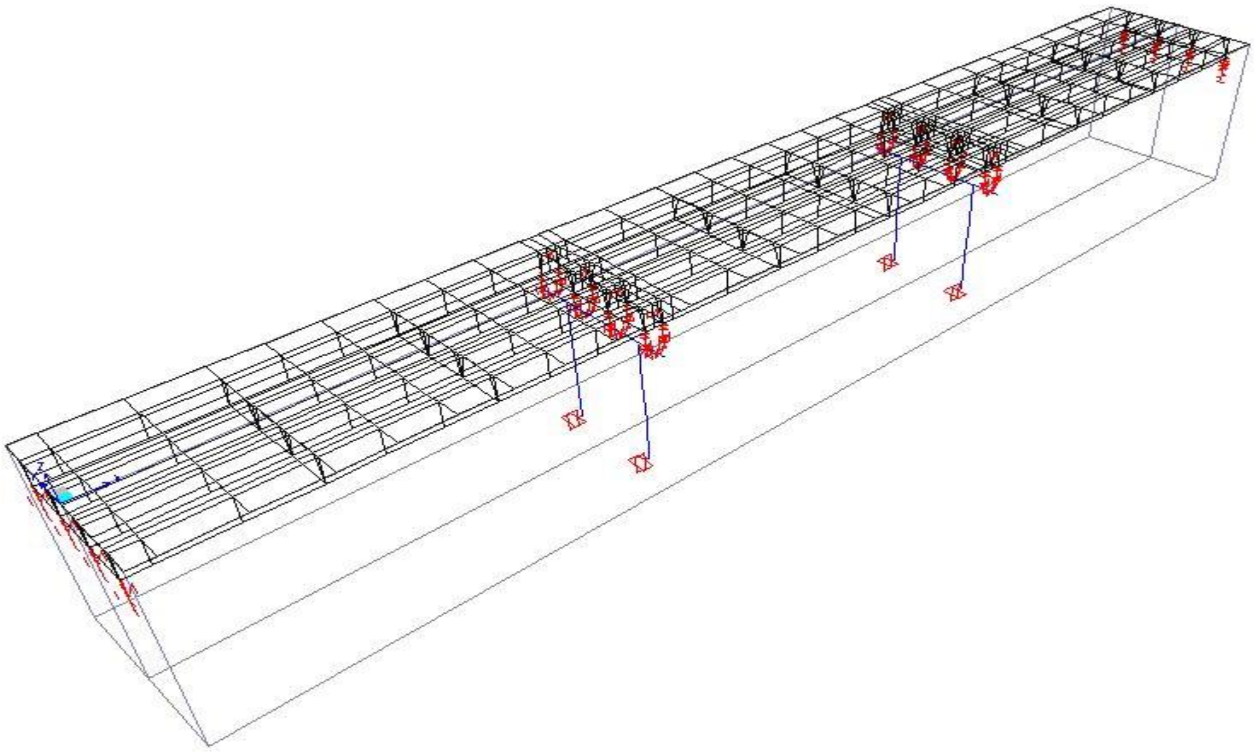
(a)



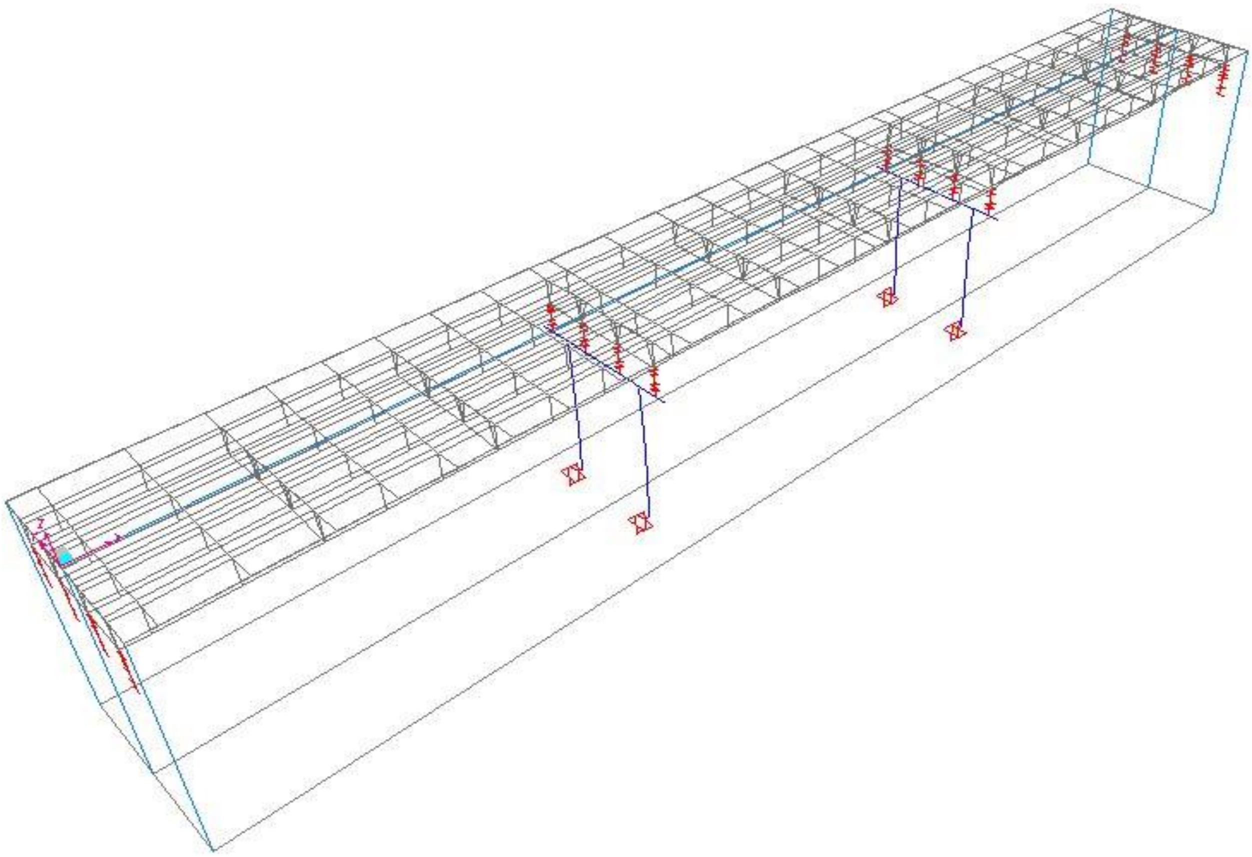
(b)

Figure 4.8 Boundary Conditions for (a) Continuous Bridge (b)Simply supported

Having the geometric and material properties specified above and loading to the software, the *SAP2000V14* structural analysis software generates the bridge models that have a close feature with the actual bridge behaviour. The finite element model of the simply supported and the continuously supported bridges are shown in *Figure 4.8* and *Figure 4.9.*

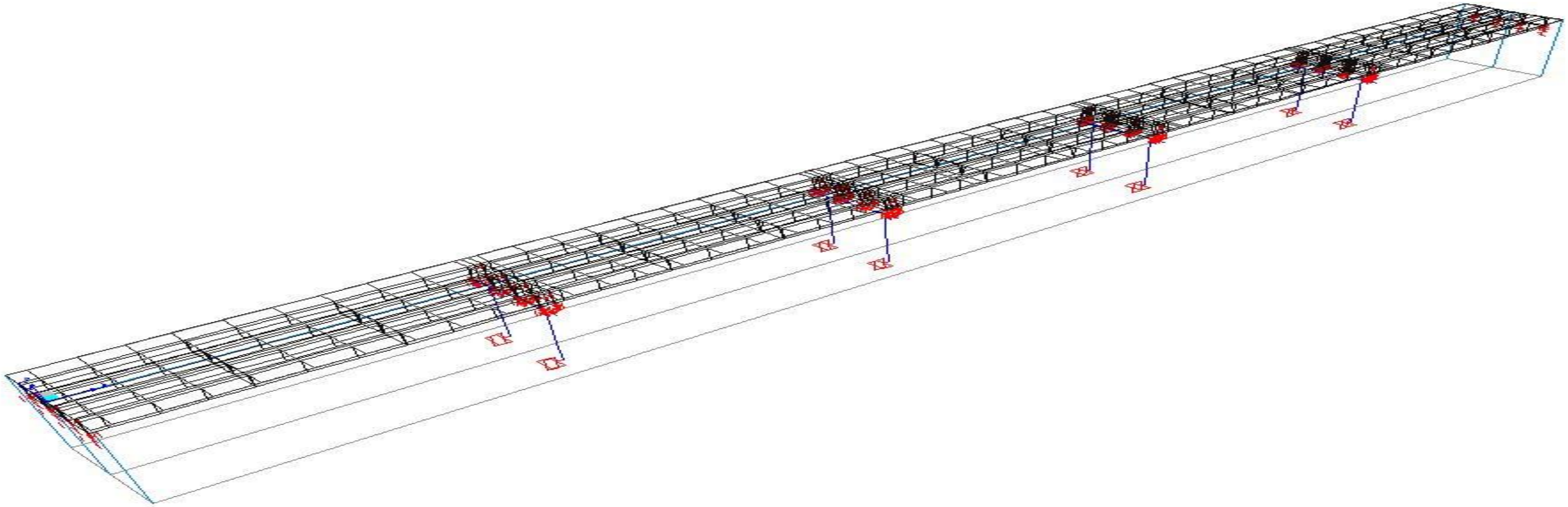


(a)

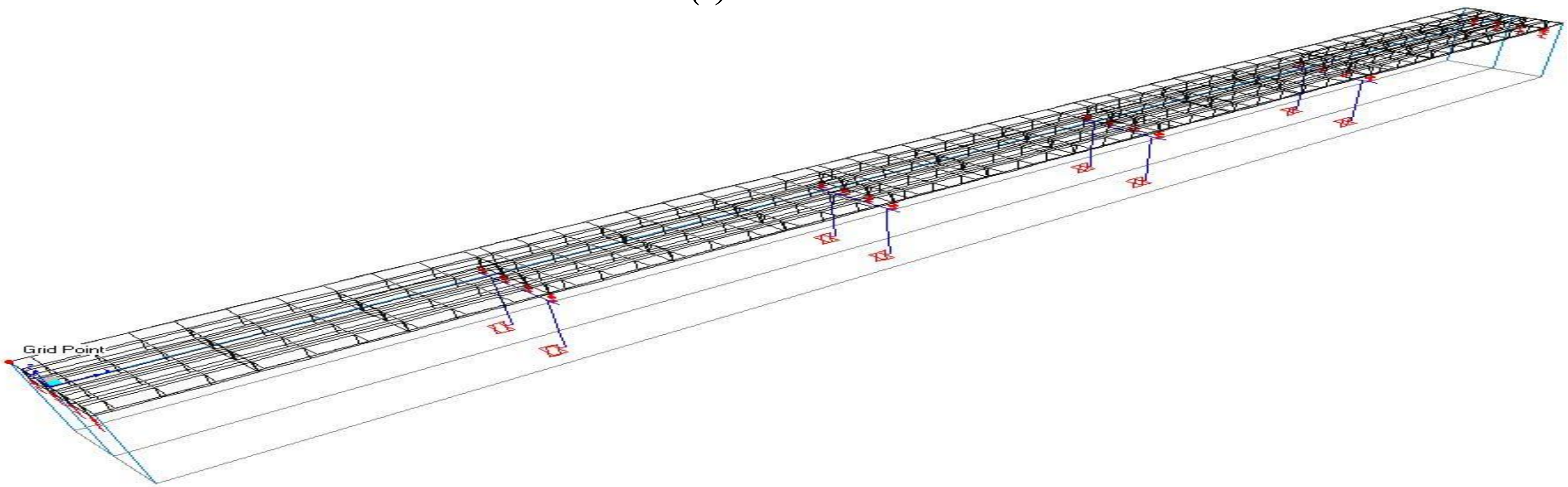


(b)

Figure 4.3 Finite element model of Bridge-1 for (a) simply supported and (b) Continuous deck-girder



(a)



(b)

Figure 4.10 Finite element model of Bridge-2 for (a) Simply supported and (b) Continuous deck-girder

4.3 Modal Response Spectrum Analysis

The seismic assessment or design of bridges is generally based on extreme or maximum dynamic response quantities and doesn't necessarily require a complete time history response. Analysis models used for modal spectral analysis are linear elastic models based on effective stiffness properties and on assumed equivalent viscous damping ratios. With these requirements response spectrum analysis can be performed for bridge systems which are expected to perform essentially in linear elastic range. Hence it is typically sufficient and more convenient to determine maximum modal response quantities by means of response spectra, once the dynamic response characteristics in the form of natural periods of vibration and mode shapes have been determined *(M.J.N. Priestley et al., 1996)*.

Therefore, response spectrum analysis is concerned with procedures to compute the peak response of a structure during an earthquake directly from design spectrum without the need for response history analysis of the structure.

The ground motion hazard (response spectrum analysis) could be carried out by using different analysis programs. *SAP2000* structural analysis software is the most appropriate program that has been utilized by several engineers. The analysis might be performed by defining the bridge location, the user can input any user defined response spectrum file from seismic hazard map of the country. The ground motion hazard and the soil site classification are the main parts of the user input data. The recently adopted "AASHTO guide Specification for the LRFD seismic bridge design" incorporates hazard maps based on a 1000-year return period.

Therefore, the seismic demands of these bridges are evaluated using a ground motion of 0.1g, from developed response spectrum data of "*EBCS8"*. This suite of ground motions is applied to the multi span girder bridges in both principal orthogonal axes, longitudinal and transverse. The soil site condition at the bridges location is considered as deep cohesion less or stiff clay soils, which is soil type III. Furthermore, 5% of structural damping is taken as energy dissipating capacity of the case study bridges.

Modal analysis which is a pre-requisite to response spectrum analysis is the analysis procedure employed to this study. As provided in various research outputs, the goal of modal analysis in structural mechanics is to determine the natural mode shapes and natural periods of the bridge or structure during vibration.

Generally the analysis models are developed to quantify the seismic demand based on approximate member dimensions from a preliminary design, estimated effective section properties and material characteristics. To capture the seismic demand, models representing the entire structural system are developed and linear elastic analysis is performed to provide quantification of member forces, stresses, displacements, and modal behaviours. The basic differential equation of dynamic analysis which is given in *Equation 4 .1* can be reduced in to an eign value problem. This basic equation, which is solved using the computer program, in a typical damped modal analysis is the eigen value problem as given below:

$$[K]\{\phi_i\} = \omega_i^2[M]\{\phi_i\} \text{……………………………………………………………} \quad (4.1)$$

Where; $[K]$ = stiffness matrix, $\{\phi_i\}$= mode shape vector (eigenvector) of mode i,

ω_i^2= natural frequency of mode i, the eigen value and $[M]$ = mass matrix

The behaviour of individual elements is described by element stiffness matrices which are square matrices of order equal to the number of nodes in the elements multiplied by the number of degrees of freedom at each node. The element stiffness matrices are functions of the geometric and material properties of the elements. Once formed, the individual element stiffness matrices are compiled into a stiffness matrix for the whole structure in a process called assembly. The resulting stiffness matrix is a square matrix of order equal to the number of degrees of freedom in the entire structure and is represented by [K].

The mass matrix [M] may be formulated to be 'consistent' or 'lumped'. A consistent formulation approximates a continuous distribution of mass throughout the elements by using their shape functions. This results in a mass which contains off-diagonal elements. In this study it is clear that the finite element program performs the consistent formulation represents better the continuous nature of real structures whereas the lumped formulation may save analysis time especially for manual calculation.

Independent of the specific dynamic input, the analysis of each bridge systems are represented within the elastic range by dynamic response modes. Typically referred to as the natural modes of vibration, characterized by independent mode shapes, ϕ_n with corresponding period of vibration, T_n.

The seismic responses of the entire bridge structures are analysed by the bridge analysis program using the response spectrum function defined from the design spectrum. The number of characteristic mode shapes and vibration periods of a bridge model depend on the selected number of dynamic degrees of freedom defined during the analytical model discretization.

Consequently, the modal analysis is conducted according to "*AASHTO LRFD* bridge design manual", in which the number of modes included should be at least three times the number of spans in the model. Generally the governing dynamic responses of bridges untypically captured by the contribution of limited number of vibration modes. Hence the fundamental or lowest mode of vibration can often provide a good indication of the dynamic response of a bridge.

This study provides some of the contributing mode shapes with corresponding natural period of vibration and the amplitude for different modes of vibration have been shown in the analysis result section. Consequently, it must be ensured that the total number of modes extracted should be enough to characterize the structure's response in the frequency range of interest. One should check the total mass participation to ensure that an adequate number of modes are included in the modal analysis.

Upon the completion of the analysis, the response spectrum analysis procedure provides maximum responses of the structure when it is vibrating in each of its significant modes. However, because these maximum modal responses will not occur at the same time during earthquake ground motion, it is necessary to use approximate procedure to estimate the maximum composite response of structure. Such procedures are typically based on an approximate combination of maximum individual modal responses.

Consequently, the seismic responses are estimated by combining the respective modal response quantities (moment, force and displacement) from individual modes by complete quadratic combination *(CQC)* method. Member forces and displacements obtained using *CQC* combination method are generally adequate for most bridge systems *(AASHTO LRFD Bridge Design Manual, 2005).* Therefore, a simple and accurate modal combination approach that satisfies the requirement of this study is *CQC* method. This method of combination has found wide acceptance by most engineers and it has been also incorporated as an option in *SAP2000* computer program for seismic analysis.

The response spectrum load case, which is the load case employed in the seismic analysis, produce three different responses; *RS-X*, *RS-Y* and *RS-XY*. The first two response spectrum load cases apply the dynamic loads along the *X* and *Y* directions. The *X* direction is defined as the longitudinal loading direction that is chosen to be from the start abutment to the end abutment, whereas the *y* direction is defined as the transversal loading direction. These elastic seismic responses on each of the principal axes of a component resulting from analysis in the two perpendicular directions shall be combined.

Hence the third response spectrum load case, *RS-XY,* uses a directional combination option with a scale factor of 30%. This response spectrum load case will satisfy the "*AASHTO* seismic guide specification". This requires the response spectrum loads to be combined using 100% of the absolute value of force effects in one of the perpendicular directions combined with 30% of the absolute value of force effects in the second perpendicular directions. These combined bridge responses are presented in tabular format in the results section.

CHAPTER V

RESULTS AND DISCUSSIONS

5.1 Introduction

In this study a suitable model for modal response spectrum analysis was developed to assess seismic behaviour of the selected simply supported and continuous girder bridges. The assessment was done by determining the lateral deformations, mode shapes and force demand at the bottom of circular piers for the selected bridges. In order to assess the contribution of higher modes in seismic response, modal analysis was used as analysis method for the dynamic problem. The number of modes in the analysis depends on the number of dicretizatin points. Only the lowest fundamental modes are physically meaningful, but in this case study the responses of selected fundamental modes were presented for the purpose of illustration.

From the modal analysis of the selected bridges, mode shapes, natural periods and longitudinal and transversal displacements for the two selected scenarios of bridge support conditions are obtained. And the other responses are the force and moment demand developed at the foot of piers. Having this response parameters comparison of seismic response of the span simply supported and the continuous girder bridges were made.

Consequently, the basic modal properties for the subject bridges are presented in this study. It is noted that the fundamental mode for the multi span simply supported bridges is transversal mode, whereas for continuous girder bridge types is a longitudinal mode. Furthermore, with all other modelling parameters being similar, the continuity of the continuous girder bridges cause to be stiffer and thus have a shorter period than that of the multi span simply supported girder bridge. Generally, the seismic analysis results and discussions over the case study bridges are briefly presented in the following sections.

5.2 Seismic Responses of the Simply Supported Bridges

The seismic responses of these bridges are evaluated using a ground motion of 0.1g, from developed design spectrum data of *EBCS8*. This ground motions is applied to the multi span simply supported girder bridges in both principal orthogonal axes, longitudinal and transversal. The selected fundamental responses for these bridge subjected to the previously defined response spectrum function is presented in this section. Furthermore, the basic modal properties of the subject bridges for simply supported case were presented in this section.

The response spectrum load case produces three different responses; *RS-X*, *RS-Y* and *RS-XY*. The first two response spectrum load cases apply the dynamic loads along the *X* and *Y* directions. The *X* direction is defined as the longitudinal loading direction that is chosen to be from the start abutment to the end abutment, whereas the *y* direction is defined as the transversal loading direction. These elastic seismic responses on each of the principal axes of a component resulting from analysis in the two perpendicular directions are combined as *RS-XY*. This combination provides two governing load cases by using a directional combination option with a scale factor of 30%.

For the first response spectrum load case loads are combined using 100% of the longitudinal force effects with 30% of the transversal force effects. This load case provides a longitudinal displacement of 126 mm for the first bridge and 176mm for the second bridge. The second load case loads are combined using 100% of the transversal force effects with 30% of the longitudinal force effects. Similarly this load case provides a transversal displacement of 160 mm for the first bridge and 196 mm for the second bridge. From the result it is noted that the fundamental mode for the two simply supported bridges showed the domination of transversal loading.

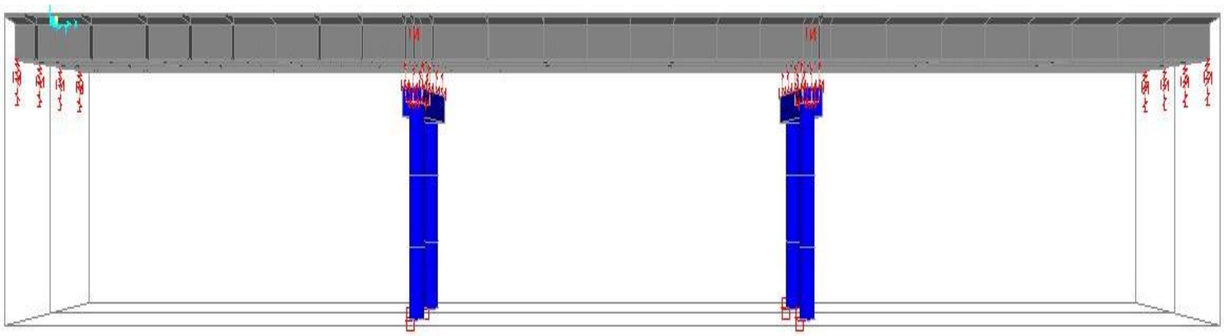
Consequently, from the governing response spectrum load cases, the displacement demand with the corresponding modal properties and mode shapes of the first bridge are presented in *Table 5.1* and *Fig. 5.1*, respectively. The number of modes that should be considered in order to obtain the seismic responses could be taken regarding the requirement that the summation of mass participation in all modes should converge with the entire bridge mass.

The number of characteristic modes in this bridge model depends on the selected number of dynamic degrees of freedom defined during the analytical model discretization. Generally it is found that, the governing dynamic responses of bridges are captured by the contribution of limited number of vibration modes. Hence for this bridge eighteen modes are carried out in the modal analysis, while the first fifteen are presented for the purpose of illustration.

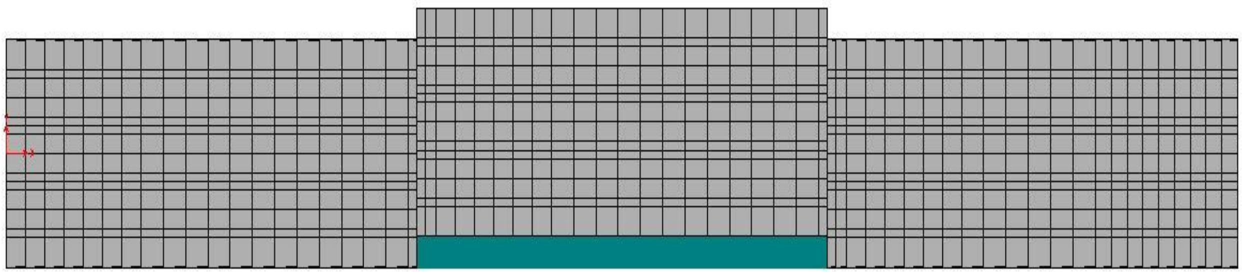
Table 5.1 Seismic Responses of the First Simply Supported Bridge

Output Case	**Mode** (No)	**Period,** T_n (Sec)	**Dampig Ratio,** ξ	**Longitudinal Response**			**Transversal Response**		
				Amplitude (mm)	Mass Particip. (%)	Sum (%)	Amplitude (mm)	Mass Particip. (%)	Sum (%)
Response Spec.	1	3.28	0.05	4.30E-01	3.00E-1	3.0E-01	**-160.1068**	30.4	30.41
Response Spec.	2	2.33	0.05	3.80E-01	4.00E-01	7.0E-01	-0.84734	0.00003	30.41
Response Spec.	3	1.46	0.05	**-126.97**	97.39	**98.01**	-2.9E-01	5E-3	30.41
Response Spec.	4	0.55	0.05	7.70E-05	1.00E-10	98.01	51.566166	52.1	**82.52**
Response Spec.	5	0.46	0.05	4.60E-04	4.00E-08	98.01	-0.253746	0.012	82.53
Response Spec.	6	0.44	0.05	-5.1E-05	1.00E-09	98.01	4.6938184	16	98.58
Response Spec.	7	0.2	0.05	1.30E-04	1.00E-08	98.01	0.7637018	0.5	99.14
Response Spec.	8	0.2	0.05	5.30E-04	8.00E-06	98.01	-2.99E-07	4E-12	99.14
Response Spec.	9	0.19	0.05	7.80E-02	0.113	98.19	1.078E-07	1E-13	99.14
Response Spec.	10	0.15	0.05	-9.6E-06	2.00E-09	98.19	7.689E-08	2E-13	99.14
Response Spec.	11	0.14	0.05	2.80E-04	4.20E-06	98.19	6.101E-07	2.2E-11	99.14
Response Spec.	12	0.12	0.05	1.90E-07	3.00E-12	98.19	0.0098044	0.0055	99.15
Response Spec.	13	0.12	0.05	1.40E-07	1.20E-12	98.19	-0.004318	0.0014	99.15
Response Spec.	14	0.09	0.05	8.70E-08	6.60E-13	98.19	0.0041148	0.0019	99.15
Response Spec.	15	0.08	0.05	2.20E-02	0.26	98.45	2.982E-07	7.7E-12	99.15

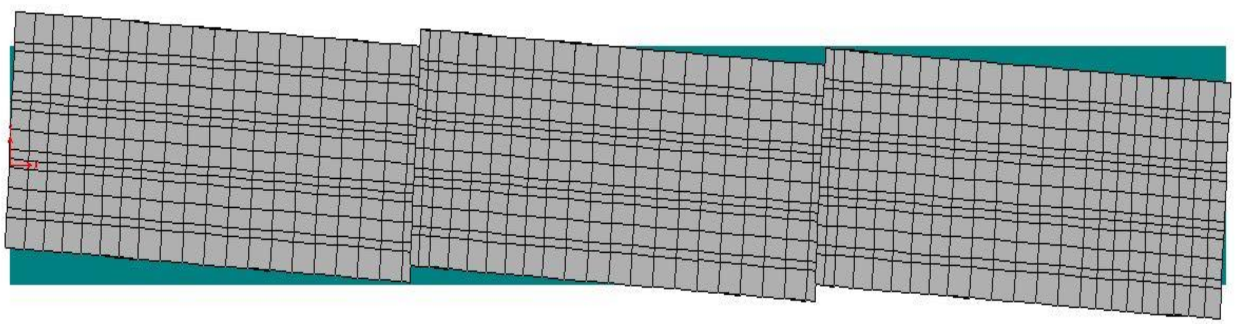
Though eighteen modes are carried out in the modal analysis and responses of fifteen of them are presented for illustration, only four of the fundamental modes are significant in seismic response. Hence the first four fundamental mode shapes of the subject bridge under transversal loading are presented in *Fig. 5.1*.



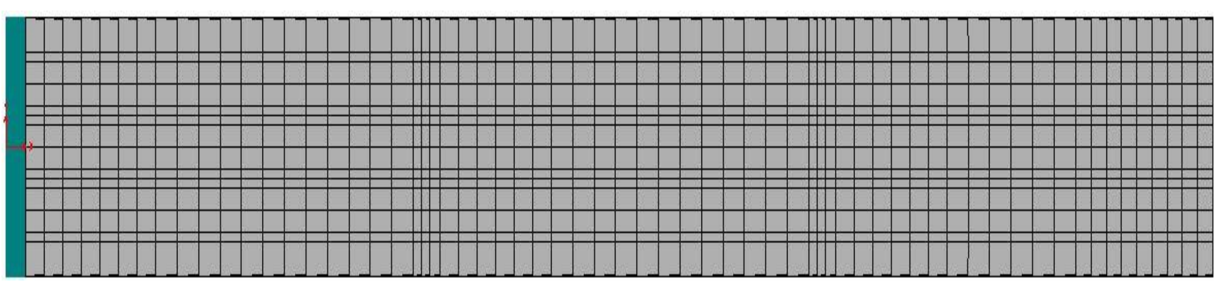
(a) Longitudinal FEM Model of the First Simply Supported Bridge



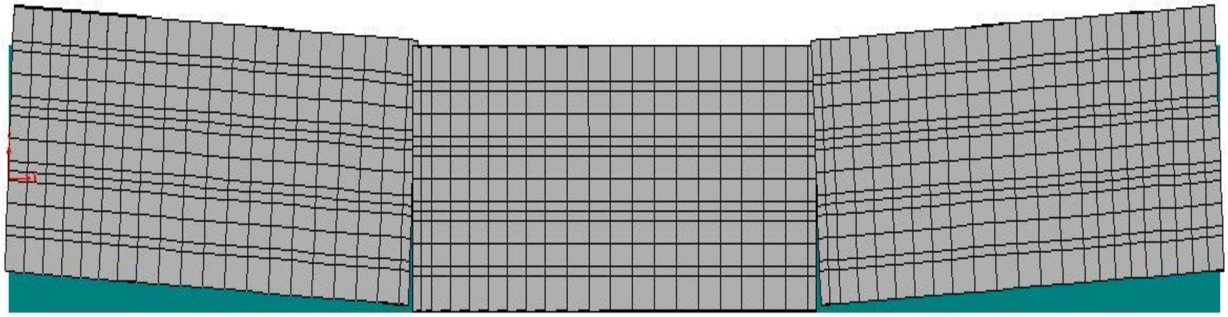
(b) First Mode-Transversal Response



(c) Second mode- Torsional Response



(d) Third Mode- Longitudinal Response



(e) Fourth Mode-Transverse and Torsinal Response

Figure 5.1 Fundamental Mode Shapes of the First Simply supported Bridge

One could note from the mode shapes in *Fig. 5.1* that, the fundamental response of the bridge is lateral translation of individual spans rather than vertical or bending mode, where all three decks move in phase and experiencing some pounding of the end spans while the middle span goes through a significantly larger translation. Under transverse load case it is found that although the transverse displacements of the end spans are relatively small and equal, while the middle span goes through a larger translation giving displacement as high as 160 mm. Consequently, under longitudinal load case it is found that the entire bridge goes through a rigid translation giving a maximum displacement of 126 mm. Therefore, from the result it is found that responses for transverse load case are dominant. The transversal response became significant because of the fundamental modal mass participation under such loading is only a single span mode where the middle span goes through a larger translation.

The deformations for all bearings at the abutments and bents that resist seismic loads are reported in the following *Table 5.2*. The overall responses of the steel bearings located at both abutments appear to be very similar regarding the fact that both are expansion bearing. Just as with the decks, the bearings located under the end spans do not deform much under the transverse load yet all bearings under the middle span deform over 150 mm. Hence, the bearings are quite flexible and do not transfer forces easily to the piers and do not place significant demand on the bridge piers in either the longitudinal or transverse directions.

Table 5.2 Deformation Demand of Support Bearings for Bridgge-1

Span Name	**Station** (m)	**Location**	**Bearing Name**	**Longit.** (mm)	**Transv.** (mm)	**Vertical** (mm)	**R1** (Deg)	**R2** (Deg)	**R2** (Deg)
Start Abutment	0	Abutment	1	2.9E-08	15.2554	0.1803	5.72E-03	7.6E-04	1.6E-11
Start Abutment	0	Abutment	2	8.7E-09	15.5444	0.1765	5.73E-03	7.4E-04	1.4E-11
Start Abutment	0	Abutment	3	8.7E-09	15.2554	0.1765	5.73E-03	7.4E-04	1.4E-11
Start Abutment	0	Abutment	4	2.9E-08	15.5444	0.1803	5.72E-03	7.6E-04	1.6E-11
Span1	21	Before Bent	1	2.1E-08	71.25	1.9E-09	3.95E-04	0.03	1.7E-04
Span1	21	After Bent	1	8.2E-08	151.223	0.0188	5.40E-03	0.031	1.4E-10
Span1	21	Before Bent	2	2.2E-07	71.25	1.5E-09	3.11E-04	0.03	9.4E-05
Span1	21	After Bent	2	2.1E-07	151.223	0.0063	5.35E-03	0.031	3.6E-10
Span1	21	Before Bent	3	2.2E-07	71.25	1.5E-09	3.11E-04	0.03	9.5E-05
Span1	21	After Bent	3	2.1E-07	151.223	0.0063	5.35E-03	0.031	3.6E-10
Span1	21	Before Bent	4	2.1E-08	71.25	1.9E-09	3.96E-04	0.03	1.7E-04
Span1	21	After Bent	4	8.2E-08	71.25	0.0188	5.40E-03	0.031	1.4E-10
Span2	42	Before Bent	1	8.3E-08	151.223	0.0189	5.43E-03	0.031	1.9E-10
Span2	42	After Bent	1	2.3E-08	71.25	1.9E-09	3.95E-04	0.03	1.6E-04
Span2	42	Before Bent	2	2.1E-07	150.153	0.0063	5.38E-03	0.031	4.4E-10
Span2	42	After Bent	2	2.5E-07	71.25	1.5E-09	3.09E-04	0.03	1.1E-04
Span2	42	Before Bent	3	2.1E-07	151.223	0.0063	5.38E-03	0.031	4.5E-10
Span2	42	After Bent	3	2.5E-07	71.25	1.5E-09	3.11E-04	0.03	1.1E-04
Span2	42	Before Bent	4	8.4E-08	150.153	0.019	5.43E-03	0.031	2.2E-10
Span2	42	After Bent	4	2.3E-08	150.153	1.9E-09	3.95E-04	0.03	1.6E-04
Span To End Abut.	63	Abutment	1	2.9E-08	15.2554	0.1804	5.75E-03	7.6E-04	1.4E-11
Span To End Abut.	63	Abutment	2	8.8E-09	15.5444	0.1765	5.76E-03	7.4E-04	1.2E-11
Span To End Abut.	63	Abutment	3	8.8E-09	15.2554	0.1765	5.76E-03	7.4E-04	1.2E-11
Span To End Abut.	63	Abutment	4	2.8E-08	15.5444	0.1804	5.75E-03	7.6E-04	1.4E-11

Consequently, the basic difference in seismic response between simply supported and continuous girder bridge is the demand in the columns. These responses are estimated by combining the respective modal response quantities. Column responses are given in terms of displacement, moment and force demands developed at the bottom of circular piers. As seen in *Table 5.3* below, higher demand of moment is resulted in transversal direction than that of the longitudinal direction, while the longitudinal loading demand is only about one-third of the transversal, indicating that as far as the columns are concerned transversal loading dominates the response. Generally, the bearings at bridge support are quite flexible that give a significant deformation and do not transfer forces easily to the piers and abutments and do not place significant demand on the piers in neither of the principal directions.

Table 5.3 Force Demand of the First Simply Supported Bridge Piers

Design Request	**Bridge Object Name**	**Station** (m)	**Column** (No)	**Location**	**P** (KN)	**V2** (KN)	**V3** (KN)	**M2** (KN-m)	**M3** (KN-m)
DReq1	Br-1 Bent-1	21	1	Bottom	1478.92	946.66	611.25	1939.56	699.41
DReq1	Br-1 Bent-1	21	2	Bottom	1478.92	946.66	611.26	1939.56	699.42
DReq1	Br-1 Bent-2	42	1	Bottom	1476.63	946.66	610.71	1939.56	699.48
DReq1	Br-1 Bent-2	42	2	Bottom	1476.64	946.66	610.71	1939.56	699.49

Similarly with the first three spanned simply supported bridge, the following section has presented the seismic response of the second five spanned simply supported bridge. These bridges differ only by structural configuration, where the first bridge is three spanned while the second one is five spanned. The bridges have quite similar loading exposure, modelling and analysis procedure as well as similar seismic responses regarding proportionality of section properties.

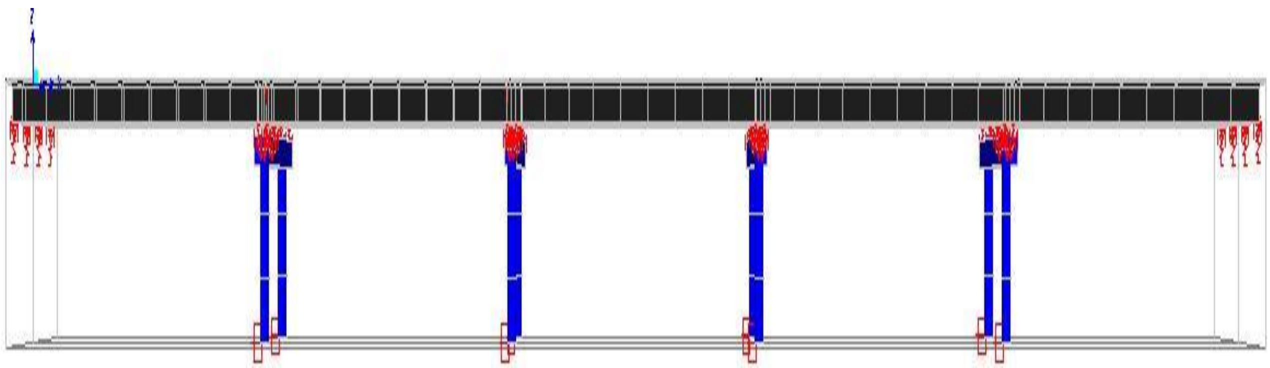
The response spectrum analysis for this bridge also provides two governing load cases. These load cases are obtained from directional combination with a scale factor of 30%. Consequently, from the governing response spectrum load cases, the displacement demand with the corresponding modal properties and mode shapes of the second bridge are presented in *Table 5.4* and *Fig. 5.2*, respectively. The number of modes that should be considered in order to obtain the seismic responses could be taken regarding the requirement that the summation of mass participation in all modes should converge with the entire bridge mass.

The number of characteristic modes of this bridge model depends on the selected number of dynamic degrees of freedom defined during the analytical model discretization. Generally it is found that, the governing dynamic responses of bridges are obtained by the contribution of limited number of vibration modes. Hence for this bridge thirty modes are carried out in the modal analysis, while the first fifteen are presented for the purpose of illustration.

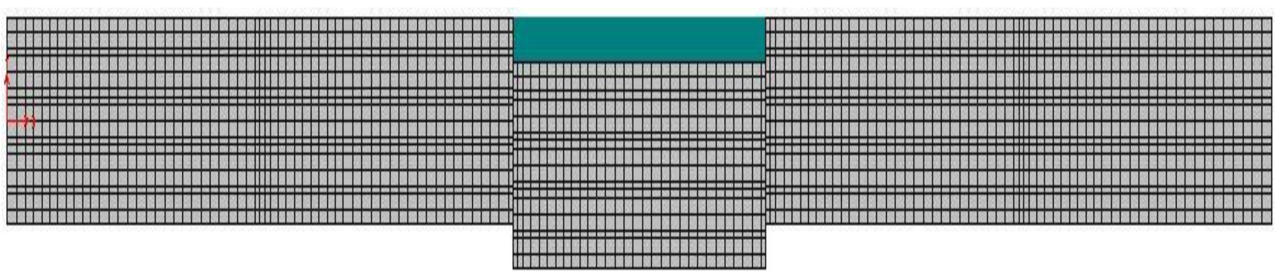
Table 5.4 Seismic Responses of the second simply Supported Bridges

Output Case	**Mode (No)**	**Period, T_n (Sec)**	**Damping Ratio,ξ**	**Longitudinal Response**			**Transversal Response**		
				Amplitude (mm)	Mass Particip. (%)	Sum (%)	Amplitude (mm)	Mass Particip. (%)	Sum (%)
Response Spec.	1	3.60	0.05	2.10E-01	5.00E-02	5.0E-01	**196.77**	18.16	18.16
Response Spec.	2	2.53	0.05	-4.80E-01	2.00E-01	2.5E-01	94.97	10.45	28.61
Response Spec.	3	2.32	0.05	6.90E-01	1.00E-01	3.5E-01	116.51	36.13	64.74
Response Spec.	4	1.79	0.05	**-176.26**	97.56000	**97.910**	9.0E-05	2.6E-11	64.74
Response Spec.	5	1.53	0.05	-7.70E-05	3.00E-11	97.910	39.04	9.31	74.05
Response Spec.	6	0.57	0.05	1.00E-04	1.00E-10	97.910	-6.26	0.46	74.5
Response Spec.	7	0.55	0.05	-5.80E-04	1.00E-08	97.910	0.3	0.0041	74.51
Response Spec.	8	0.52	0.05	-2.50E-04	1.00E-08	97.910	-9.92	20.37	**94.88**
Response Spec.	9	0.51	0.05	1.80E-04	7.00E-09	97.910	-3.93	3.49	98.37
Response Spec.	10	0.48	0.05	1.00E-04	3.00E-09	97.910	1.5	0.62	98.99
Response Spec.	11	0.26	0.05	3.10E-05	6.60E-10	97.910	0.39	0.055	99.04
Response Spec.	12	0.25	0.05	-4.40E-02	2.50E-06	97.910	-4.0E-06	4.8E-11	99.04
Response Spec.	13	0.24	0.05	1.00E-01	0.0580	97.970	5.6E-08	8.7E-15	99.04
Response Spec.	14	0.23	0.05	6.10E-02	0.0240	97.990	4.9E-07	1.8E-12	99.04
Response Spec.	15	0.22	0.05	-8.50E-03	1.20E-07	97.990	-5.0E-07	1.5E-12	99.04

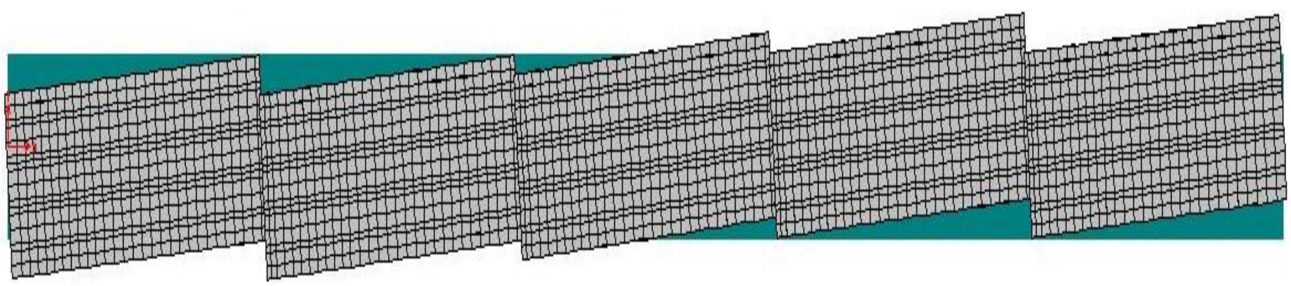
Though thirty modes are carried out in the modal analysis and responses of fifteen of them are presented for illustration. Four of the fundamental modes are significant in seismic response longitudinal loading while the first eight modes contribute for seismic response of the bridge under transversal loading. One could note that the transverse loading governs the bridge response; however the first eight fundamental mode shapes of the subject bridge under such loading are presented in *Fig. 5.2.*



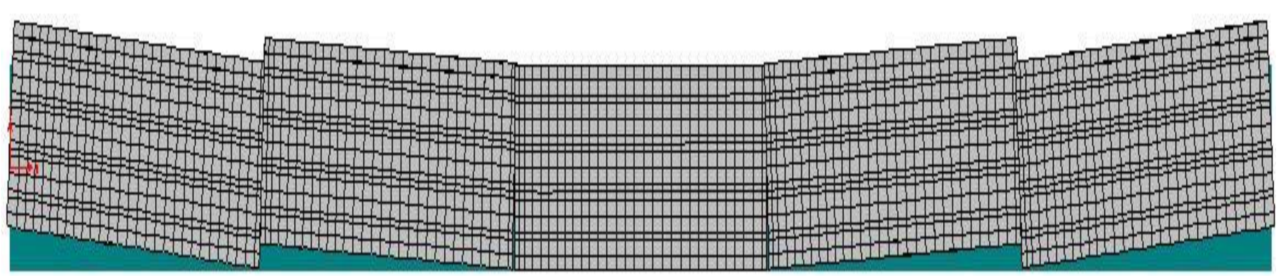
(a) Longitudinal FEM Model of the SecondSimply Supported Bridge



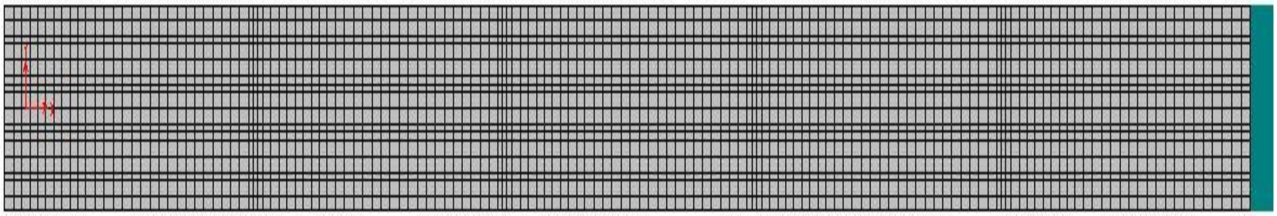
(b) First Mode-Transversal Response



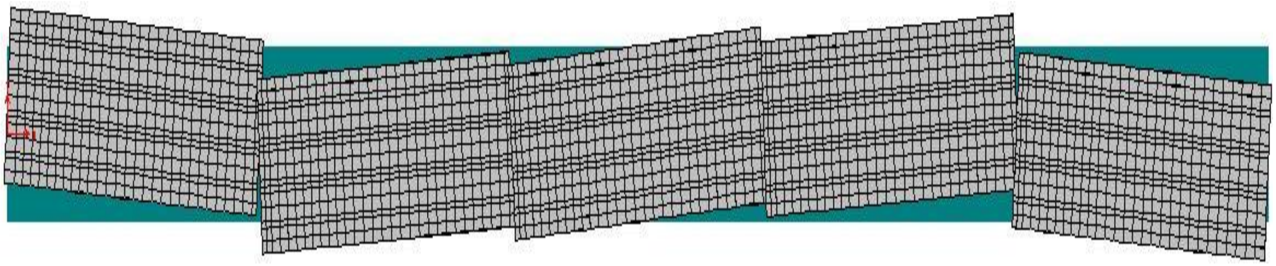
(c) Second mode- Torsional Response



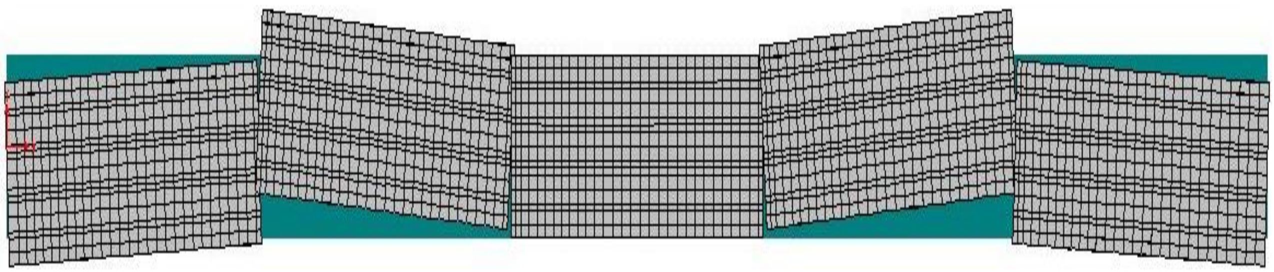
(d) Third Mode-Transverse and Torsinal Response



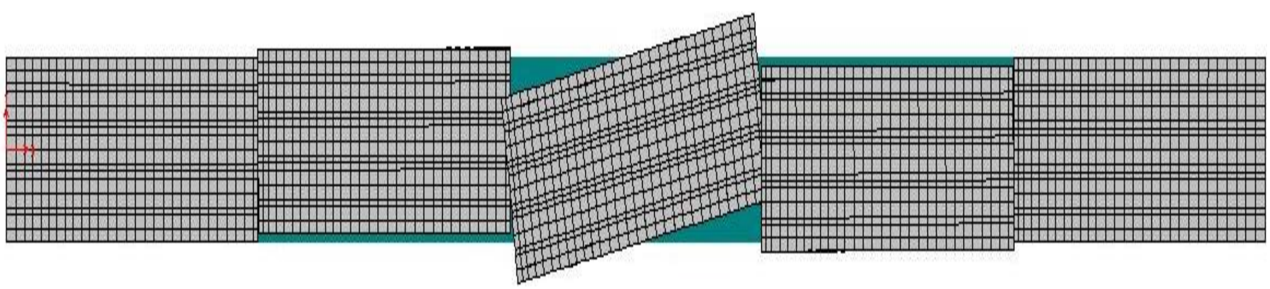
(e) Fourth Mode- Longitudinal Response



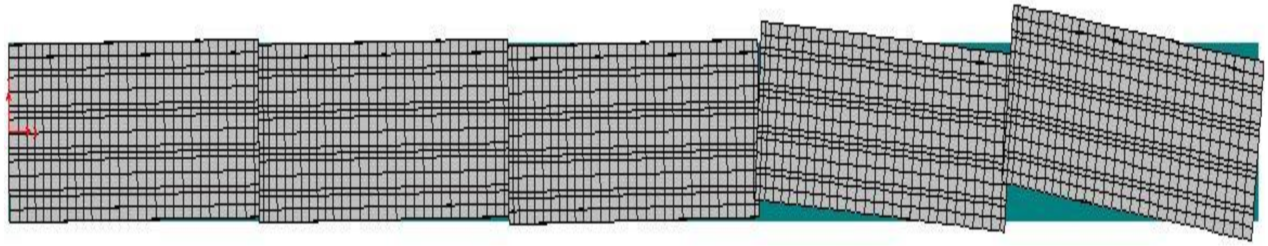
(f) Fifth Mode-Torsional Response



(g) Sixth Mode-Transverse and Torsinal Response



(h) Seventh Mode-Transverse and Torsinal Response



(i) Eighth Mode-Transverse and Torsinal Response

Figure 5.2 Fundamental Mode Shapes of the Second Simply supported Bridge

Just as the first bridge one could also note from the mode shapes in *Fig. 5.2* that, the fundamental response of the second bridge is lateral translation of individual spans rather than vertical mode, where all the five decks move in phase and experiencing some pounding of the end spans while the middle span goes through a significantly larger translation at its fundamental lowest mode. Under transverse load case it is found that although the transverse displacements of the end spans are relatively small and equal, while the middle span goes through a larger translation giving displacement as high as 196 mm. Consequently, under longitudinal load case it is found that the entire bridge goes through a rigid translation giving a maximum displacement of 176 mm. Therefore, from the result it is found that responses for transverse load case are dominant. The transversal response became significant because of the fundamental modal mass participation under such loading is only a single span mode where the middle span goes through a larger translation.

The deformations for bearings at the abutments and bents that resist seismic loads are reported in the following *Table 5.5*. The overall responses of the steel bearings located at both abutments appear to be very similar regarding the fact that both are expansion bearing. Just as with the decks, the bearings located under the end spans do not deform much under the transverse load yet all bearings under the middle span deform over 175 mm. Hence, the bearings are quite flexible and do not transfer forces easily to the piers and do not place significant demand on the bridge piers in either the longitudinal or transverse directions.

Table 5.5 Deformation Demand of Support Bearings for bridge-2

Span Name	**Station** (m)	**Location**	**Bearing Name**	**Longit.** (mm)	**Trans.** (mm)	**Vertical** (mm)	**R1** (Deg)	**R2** (Deg)	**R2** (Deg)
Start Abutm.	0	Abutment	1	5.1E-08	37.651	1.7E-01	0.011	6.1E-04	2.9E-11
Start Abutm.	0	Abutment	2	1.1E-08	37.651	0.1611	0.011	5.7E-04	2.6E-11
Start Abutm.	0	Abutment	3	1.1E-08	37.651	0.1611	0.011	5.7E-04	2.6E-11
Start Abutm.	0	Abutment	4	5.1E-08	37.651	0.1686	0.011	6.1E-04	3.0E-11
Span1	25.25	Before Bent	1	4.0E-08	98.723	4.8E-09	6.1E-04	0.026	2.2E-04
Span1	25.25	After Bent	1	1.1E-07	175.83	0.015	4.0E-03	0.027	3.6E-10
Span1	25.25	Before Bent	2	2.9E-07	98.723	1.4E-09	4.5E-04	0.026	1.4E-04
Span1	25.25	After Bent	2	1.8E-07	175.83	0.0052	4.1E-03	0.027	6.8E-10
Span1	25.25	Before Bent	3	2.8E-07	98.723	1.4E-09	4.5E-04	0.026	1.4E-04
Span1	25.25	After Bent	3	1.8E-07	175.83	0.0052	4.1E-03	0.027	6.8E-10
Span1	25.25	Before Bent	4	4.0E-08	98.723	4.8E-09	6.1E-04	0.026	2.2E-04
Span1	25.25	After Bent	4	1.1E-07	98.723	0.015	4.0E-03	0.027	3.6E-10
Span2	50.5	Before Bent	1	3.5E-08	175.83	3.5E-09	3.8E-04	0.027	1.5E-04
Span2	50.5	After Bent	1	7.5E-08	98.723	0.0233	6.5E-03	0.027	2.6E-10
Span2	50.5	Before Bent	2	1.9E-07	175.83	2.0E-09	2.7E-04	0.027	9.9E-05
Span2	50.5	After Bent	2	1.9E-07	98.723	0.0077	6.5E-03	0.027	4.5E-10
Span2	50.5	Before Bent	3	1.9E-07	175.83	2.0E-09	2.7E-04	0.027	9.9E-05
Span2	50.5	After Bent	3	1.9E-07	98.723	0.0077	6.5E-03	0.027	4.5E-10
Span2	50.5	Before Bent	4	3.5E-08	175.83	3.5E-09	3.8E-04	0.027	1.5E-04
Span2	50.5	After Bent	4	7.6E-08	175.83	0.0233	6.5E-03	0.027	2.6E-10

Just as the first bridge the seismic demand of the second bridge piers are estimated by combining the respective modal response quantities. Column responses are given in *Table 5.6*, in terms of displacement, moment and force demands developed at the bottom of circular piers. Similarly, higher demand of moment is resulted in transversal direction than that of the longitudinal direction, while the longitudinal loading demand is only about a third of the transversal, indicating that as far as the columns are concerned transversal loading dominates the response. Hence it could be noted that, the bearings at bridge support are quite flexible that give a significant deformation and do not transfer forces easily to the piers and abutments and do not place significant demand on the piers in neither of the principal directions.

Table 5.6 Force Demand of the First Simply Supported Bridge Piers

Design Request	Bridge Object Name	Station (m)	Column (No)	Location	P (KN)	V2 (KN)	V3 (KN)	M2 (KN-m)	M3 (KN-m)
DReq1	Br-2 Bent-1	25.25	1	Bottom	1821.82	1471.42	868.35	2918.94	1020.28
DReq1	Br-2 Bent-1	25.25	2	Bottom	1821.83	1471.42	868.35	2918.94	1020.28
DReq1	Br-2 Bent-2	50.50	1	Bottom	1911.11	1548.02	881.26	2945.27	1055.18
DReq1	Br-2 Bent-2	50.50	2	Bottom	1911.11	1548.02	881.26	2945.27	1055.18
DReq1	Br-2 Bent-3	75.75	1	Bottom	1911.11	1548.02	881.26	2948.38	1055.18
DReq1	Br-2 Bent-3	75.75	2	Bottom	1911.11	1548.02	881.26	2948.38	1055.18
DReq1	Br-2 Bent-4	101.00	1	Bottom	1821.82	1471.42	868.35	2918.94	1020.28
DReq1	Br-2 Bent-4	101.00	2	Bottom	1821.83	1471.42	868.35	2918.94	1020.28

5.2.1 Effect of Higher Modes in Seismic Response of the Simply Supported Bridges

Earthquake ground motion tend to excite the lowest modes of vibration more than the higher modes, as a result good approximation of the earthquake response of bridges can be obtained from only a few modes. This makes the modal superposition analysis a powerful tool for bridge systems, particularly when large numbers of degree of freedoms are involved *(M.J.N. Priestley et al., 1996)*. Consequently, in this study the multi degree of freedom system of bridges have eighteen modes for the three spanned bridge and thirty modes for that of the five span bridge.

According to (M.J.N. Priestley et al., 1996), effective mass or mass participation factors should be provided from modal analysis programs in order to determine how many modes should be considered for the response of bridges the sum of these mass participation factors for all modes in a given response most equal the mass of the bridge. Consequently, mass participation factors of 80 to 90% of the total bridge mass in a given response can be considered sufficient to capture the dominant dynamic response of the bridge structure.

The first fifteen seismic responses with their corresponding modal mass participations of the subject simply supported bridges are presented in *Table 5.1* and *Table 5.4*. It is noted that the fundamental modes of the bridges are transversal mode giving modal participating mass ratio of 83% for the first bridge and 95% for the second bridge. These mass participations are attained at their fourth and eighth mode, respectively. Hence, four modes for the first bridge and eight modes for the second bridge might be considered sufficient to determine the transversal responses of the subject bridges. The modal seismic response of both bridges for longitudinal loading also show that, all the deck slabs go through significant translation giving modal participating mass ratio of about 98%.

Therefore, the first bridge shows the contribution of only the first three modes in longitudinal loading and the first four modes for transversal loading whereas, for the second bridge the first four modes in longitudinal loading and the first eight modes in transversal direction should be considered to determine the dominant seismic response of the simply supported bridges. These limitations of modal contribution in the computation dynamic responses is based on where the modal mass participation factors are about to reach 80 % or above of the total bridge mass in a given response. Therefore, it is also found that the contributing higher modes should be considered to obtain the dominant dynamic response of the simply supported bridges.

5.3 Seismic Response of Continuous Girder Bridges

Just as the simply supported bridges, the seismic responses of the continuous bridges are also evaluated using a ground motion of 0.1g, EBCS-*8* response spectrum data. This ground motions is applied to the bridges in both principal orthogonal axes, longitudinal and transversal. The selected responses of these bridges subjected to the earthquake loading are presented in this section.

From the modal response spectrum analysis of the continuous girder bridges, the displacement demands with the corresponding modal properties are presented. Like the simply supported Bridges, the numbers of modes considered in this analysis are taken regarding the requirement that the summation of mass participation in all modes should converge with the entire bridge mass. The member forces and displacements are estimated by combining the respective modal response quantities.

Likewise the simply supported bridges, the response spectrum load case of the continuous bridges produces three different responses; *RS-X*, *RS-Y* and *RS-XY*. The first two response spectrum load cases apply the dynamic loads along the *X* and *Y* directions. Likewise the simply supported bridges, these elastic seismic responses are directionally combined with a scale factor of 30% to provide two governing load cases.

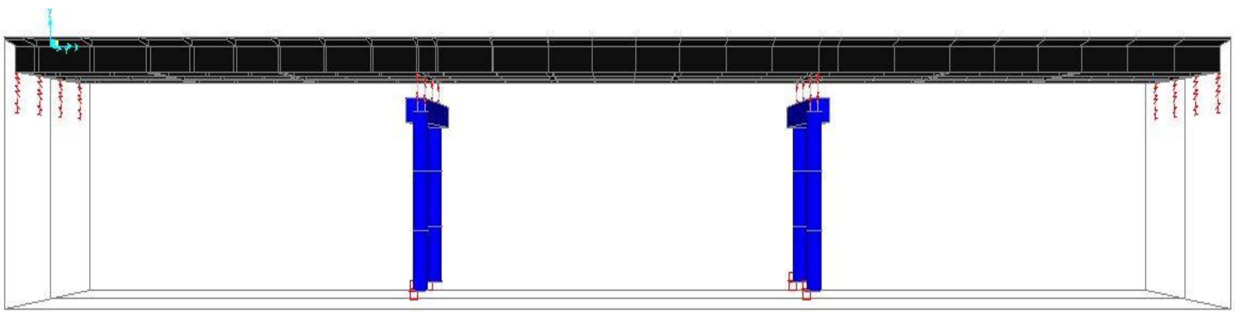
For the first response spectrum load case loads are combined using 100% of the longitudinal force effects with 30% of the transversal force effects. This load case provides a longitudinal displacement of 123 mm for the first bridge and 171mm for the second bridge. The second load case loads are combined using 100% of the transversal force effects with 30% of the longitudinal force effects. Similarly this load case provides a transversal displacement of 38 mm for the first bridge and 56 mm for the second bridge. From the result it is noted that the basic modes for the two continuous bridges showed the domination of the longitudinal loading.

Table 5.7 Seismic Responses of the First Continuous girder Bridges

Output Case	**Mode** (No)	**Period, T_n** (Sec)	**Damping Ratio, ξ**	**Longitudinal Response**			**Transversal Response**		
				Amplitude (mm)	Mass Particip. (%)	Sum (%)	Amplitude (mm)	Mass Particip. (%)	Sum (%)
Response Spec.	1	2.756	0.05	**123.52**	98.08	**98.08**	-1.2E-06	9E-15	9E-15
Response Spec.	2	1.842	0.05	0.00006	8E-09	98.08	0.16	0.0005	0.0005
Response Spec.	3	1.129	0.05	-4.2E-07	1E-14	98.08	**37.75**	98.99	**98.99**
Response Spec.	4	0.225	0.05	0.0015	0.00004	98.08	-1E-06	2E-11	98.99
Response Spec.	5	0.181	0.05	-0.0074	0.003	98.09	6.5E-07	2E-11	98.1
Response Spec.	6	0.177	0.05	3.5E-07	6E-12	98.09	0.025	0.03	99.03
Response Spec.	7	0.152	0.05	-6E-08	3E-13	98.09	-0.016	0.02	99.05
Response Spec.	8	0.146	0.05	3.1E-06	1E-09	98.09	-0.00015	0.000003	99.05
Response Spec.	9	0.132	0.05	-0.001	0.0002	98.09	-5.3E-07	5E-11	99.05
Response Spec.	10	0.121	0.05	-1.2E-07	3E-12	98.09	-0.0081	0.01	99.06
Response Spec.	11	0.096	0.05	0.035	1.1E-10	98.09	-4.2E-08	2.9E-13	99.06
Response Spec.	12	0.078	0.05	-2.6E-06	1.35	99.44	0.00043	5.7E-14	99.06

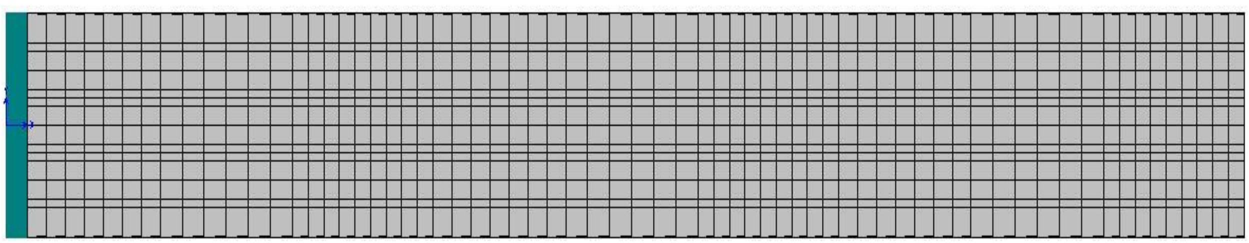
Response Spec.	13	0.076	0.05	0.021	6.4E-09	99.44	4.1E-08	4.4E-08	99.06
Response Spec.	14	0.058	0.05	6.2E-07	0.021	99.46	0.0044	5.1E-11	99.06
Response Spec.	15	0.058	0.05	-0.0023	2.1E-11	99.46	9.8E-07	0.067	99.13

In similar manner to the first simply supported bridges, the governing dynamic responses of the first continuous bridge are captured by the contribution of limited number of vibration modes. Hence for this bridge eighteen modes are carried out in the modal analysis, while the first fifteen are presented for the purpose of illustration. Though eighteen modes are carried out in the modal analysis and responses of fifteen of them are presented for illustration, only a single mode is significant in seismic response. But the first four lowest mode shapes of the subject bridge under longitudinal loading are presented in *Fig. 5.3.*

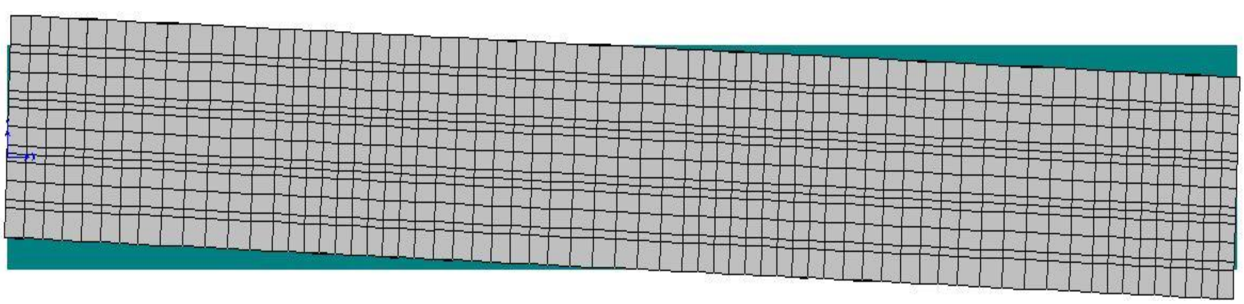


(

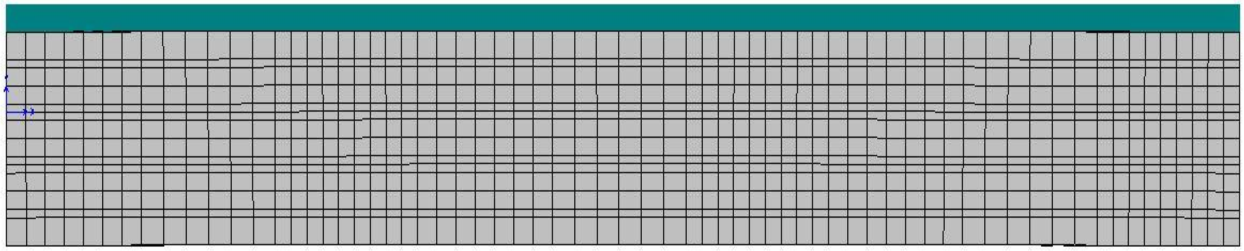
a) Longitudinal FEM Model of the First Continuous Bridge



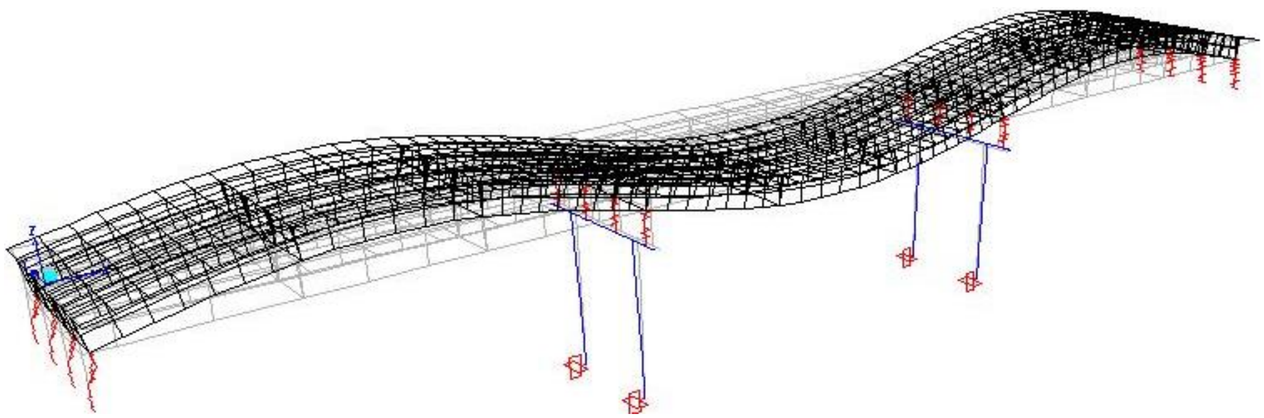
(b) First Mode- Longitudinal Response



(c) Second mode- Torsional Response



(d) Third Mode-Transversal Response



(d) Fourth Mode-Bending Response

Figure 5.3 Fundamental Mode Shapes of the First Continuous Bridge

With all other modelling parameters being equal, the continuity of the continuous girder bridge girders causes it to be stiffer and thus have a shorter period than that of the simply supported girder bridge. As expected, all decks in the multi span continuous girder bridge appear to go through a rigid translation since the bending stiffness of the deck and columns are much greater than the translational stiffness of the bearings. Therefore, from the mode shapes obtained, it is found that translational responses which are the lowest modal responses are dominant.

Unlike the simply supported case, one could note from the mode shapes in *Fig. 5.3* that, the dominant response of the continuous bridge is lateral translation of the entire bridge, giving a higher longitudinal displacement than the transverse direction. Consequently, under the governing longitudinal load case it is found that the entire bridge goes through a rigid translation giving a maximum displacement of 123 mm, while the transverse loading results 38mm of deformation which is about a quarter of that the longitudinal response. Therefore, from the result it is found that the longitudinal load case governs bridge responses.

Furthermore, the peak longitudinal displacement of the continuous girder bridge has some similarities to that of the simply supported bridges. Where an overall longitudinal displacements of continuous bridge is about 123mm while the simply supported bridge gives 126 mm.

The deformations for all bearings at the abutments and bents that resist seismic loads are reported in the following *Table 5.7*. The overall responses of the steel bearings located at both abutments appear to be very similar regarding the fact that both are expansion bearing.

Table 5.8 Deformation Demand of Support Bearings for bridge-1

Span Name	**Station** (m)	**Bearing Name**	**Longit.** (mm)	**Transv.** (mm)	**Vertical** (mm)	**R1** (Deg)	**R2** (Deg)	**R2** (Deg)
Start Abutment	0	1	16.760	5.24E+00	0.127	1.05E-02	3.52E-03	1.79E-10
Start Abutment	0	2	16.750	5.240	0.127	1.04E-02	3.29E-03	1.90E-10
Start Abutment	0	3	16.750	5.240	0.127	1.04E-02	3.29E-03	1.89E-10
Start Abutment	0	4	16.760	5.240	0.127	1.05E-02	3.52E-03	1.81E-10
Span1	21	1	13.538	6.25E-08	0.074	4.68E-03	2.80E+00	2.42E-02
Span1	21	1	13.538	3.71E-07	0.074	4.38E-03	2.80E+00	4.47E-02
Span1	21	2	13.538	3.71E-07	0.074	4.39E-03	2.80E+00	4.48E-02
Span1	21	2	13.538	6.24E-08	0.074	4.69E-03	2.80E+00	2.42E-02
Span2	42	3	13.538	6.26E-08	0.074	4.89E-03	2.80E+00	2.42E-02
Span2	42	3	13.538	3.71E-07	0.074	3.55E-03	2.80E+00	4.48E-02
Span2	42	4	13.538	3.71E-07	0.074	3.56E-03	2.80E+00	4.49E-02
Span2	42	4	13.538	6.25E-08	0.074	4.90E-03	2.80E+00	2.42E-02
Span To End Abutment	63	1	16.760	5.260	0.127	1.10E-02	3.53E-03	1.62E-10
Span To End Abutment	63	1	16.750	5.260	0.127	1.08E-02	3.29E-03	1.77E-10
Span To End Abutment	63	2	16.750	5.260	0.127	1.08E-02	3.30E-03	1.75E-10
Span To End Abutment	63	2	16.760	5.260	0.127	1.10E-02	3.53E-03	1.65E-10

Unlike the simply supported case, relatively small deflections of bearings in continuous bridges were resulting with higher seismic demand at the bottom of piers. The bearing deformation under the governing longitudinal loading of the subject bridge is about 13 mm. Hence the bearing deformations in the transverse as well in the longitudinal directions are insignificant which may contribute to the increased demand placed on the columns.

Therefore, the bearings are not flexible and they do transfer significant forces to the piers and place significant seismic demand on the bridge piers in both the longitudinal and the transverse directions.

Furthermore, the seismic responses of bridge piers are estimated by combining the respective modal response quantities. The most significant differences between the responses of the multi span simply supported and multi span continuous girder bridges appear to be in the columns. The axial forces demand for the continuous bridge piers are as high as twice of the simply supported piers. As presented on *Table 5.9*, column responses are given in terms of lateral displacements, moment and force demand developed at the foot of circular piers.

One could also note that, higher demand of shear force and moment is resulted in longitudinal direction than that of the transversal direction, while the transversal loading demand is only about sixty percent of the longitudinal; indicating that as far as the columns are concerned longitudinal loading dominates the response.

Table 5.9 Force Demand of the First continuous Girder Bridge Piers

Design Request	**Bridge Object Name**	**Station** (m)	**Column** (No)	**Location**	**P** (KN)	**V2** (KN)	**V3** (KN)	**M2** (KN-m)	**M3** (KN-m)
DReq1	Br-1 Bent-1	21	1	Bottom	3994.47	800.70	1473.1	1649.8	2696.7
DReq1	Br-1 Bent-1	21	2	Bottom	3994.47	800.70	1473.1	1649.8	2696.7
DReq1	Br-1 Bent-2	42	1	Bottom	3994.47	800.70	1475.1	1649.8	2696.7
DReq1	Br-1 Bent-2	42	2	Bottom	3994.47	800.70	1475.1	1649.8	2696.7

Similarly with the three spanned continuous bridge, the following section has presented the seismic response of the five spanned continuous bridge. These bridges differ only by structural configuration, where the first bridge is three spanned while the second one is five spanned. The bridges have quite similar loading exposure, modelling and analysis procedure as well as similar seismic responses regarding proportionality of section properties.

Just as the first bridge, the response spectrum load case of the second continuous bridge produces three different responses; *RS-X*, *RS-Y* and *RS-XY*. The first two response spectrum load cases apply the dynamic loads along the *X* and *Y* directions. These elastic seismic responses are directionally combined with a scale factor of 30% to provide two governing load cases.

Consequently, from the governing response spectrum load cases, the displacement demand with the corresponding modal properties and mode shapes of the second bridge are presented in *Table 5.10* and *Fig. 5.4*, respectively. The number of modes that should be considered in to obtain the seismic responses are taken in such a way that the summation of mass participation in all modes should converge with the entire bridge mass.

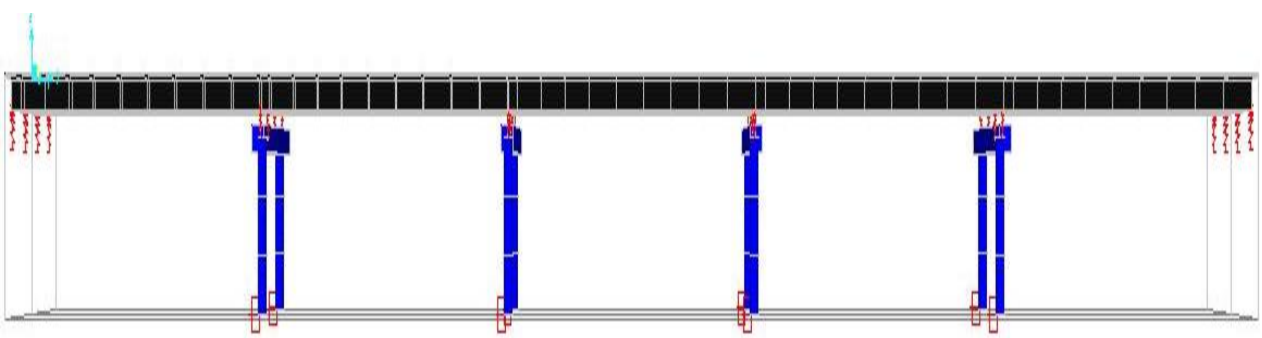
Just as the other case study bridges, the governing dynamic responses of the second continuous bridge are obtained by the contribution of limited number of vibration modes. Hence for this bridge thirty modes are carried out in the modal analysis, while the first fifteen are presented for the purpose of illustration.

Table 5.10 Seismic Responses of the Second Continuous girder Bridges

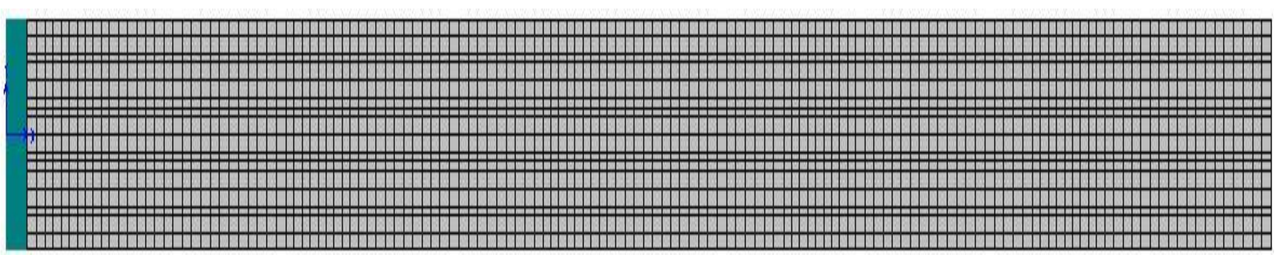
Output Case	**Mode** (No)	**Period, T_n** (Sec)	**Damping Ratio, ξ**	**Longitudinal Response**			**Transversal Response**		
				Amplitude (mm)	Mass Particip. (%)	Sum (%)	Amplitude (mm)	Mass Particip. (%)	Sum (%)
Response Spec.	1	2.54	0.05	**171.89**	97.98	**97.98**	-4.9E-09	3E-16	3E-16
Response Spec.	2	1.37	0.05	0.00033	2E-09	97.98	-0.043	0.00003	0.00003
Response Spec.	3	1.10	0.05	5.2E-08	8E-17	97.98	**56.35**	97.14	**97.14**

Response Spec.	4	0.54	0.05	-2.8E-07	2E-14	97.98	-2.7	2	98.97
Response Spec.	5	0.26	0.05	-2E-06	2E-11	97.98	-6.9E-08	1E-13	98.97
Response Spec.	6	0.23	0.05	0.0089	0.0005	97.98	-3.7E-08	8E-15	98.97
Response Spec.	7	0.23	0.05	-1.6E-05	2E-09	97.98	-1.5E-05	2E-09	98.97
Response Spec.	8	0.19	0.05	-6.1E-09	5E-16	97.98	0.004	0.0003	98.97
Response Spec.	9	0.19	0.05	9.3E-07	1E-11	97.98	1.3E-06	3E-11	98.97
Response Spec.	10	0.17	0.05	8.1E-06	1E-09	97.98	-3.6E-05	2E-08	98.97
Response Spec.	11	0.16	0.05	6.5E-08	1.4E-13	97.98	0.0074	0.0012	98.97
Response Spec.	12	0.16	0.05	-0.0031	0.0003	97.98	1.1E-08	1.9E-15	98.97
Response Spec.	13	0.14	0.05	-1.5E-07	1E-12	97.98	-2.2E-06	3.4E-10	98.97
Response Spec.	14	0.13	0.05	1.2E-06	6E-11	97.98	-1.8E-06	1.7E-10	98.97
Response Spec.	15	0.12	0.05	4.8E-08	1.7E-13	97.98	0.0065	0.002	98.97

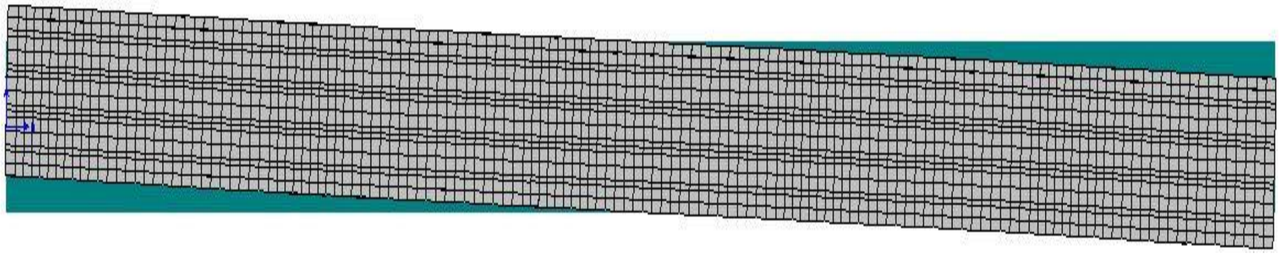
Though thirty modes are carried out in the modal analysis and responses of fifteen of them are presented for illustration, only a single mode is significant in seismic response. But the first four lowest mode shapes of the subject bridge under longitudinal loading are presented in *Fig. 5.4.*



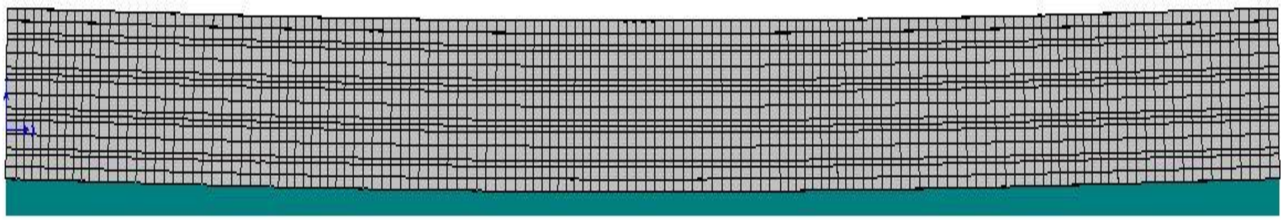
(a) Longitudinal FEM Model of the First Continuous Bridge



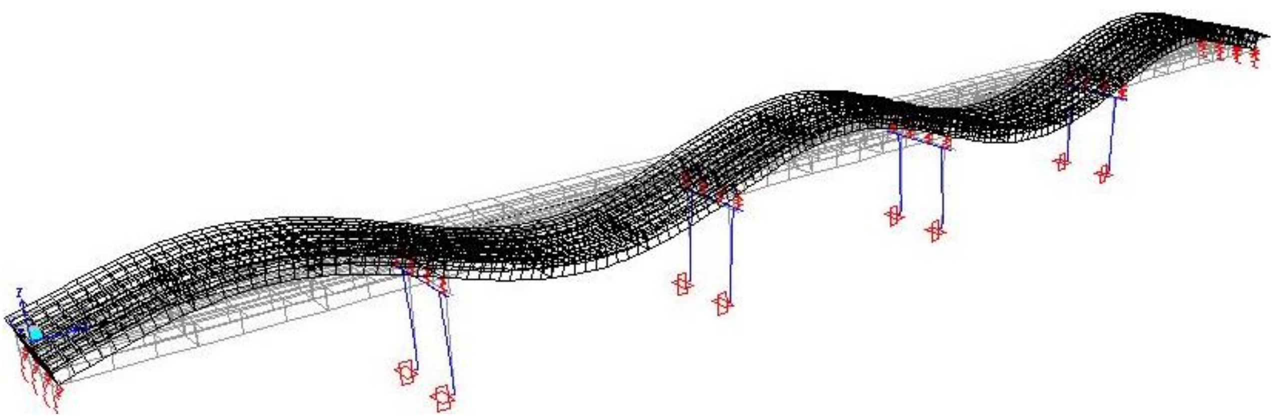
(b) First Mode- Longitudinal Response



(c) Second mode- Torsional Response



(d) Third Mode-Transversal Response



(d) Fourth Mode-Bending Response

Figure 5.4 Fundamental Mode Shapes of the Second Continuous Bridge

Likewise the first continuous bridge, the fundamental the mode shapes of the second continuous bridge is lateral translation of the entire bridge, giving a higher longitudinal displacement than the transverse direction. Consequently, under the governing longitudinal load case it is found that the entire bridge goes through a rigid translation giving a maximum displacement of 172 mm, while the transverse loading results 56 mm of deformation which is about one- third of that the longitudinal response. Therefore, from the result it is found that responses for longitudinal load case are dominant.

Furthermore, just as the first bridge, the peak longitudinal displacement of the second continuous girder bridge has some similarities to that of its counterpart simply supported bridge. Where an overall longitudinal displacements of continuous bridge is about 172 mm while the simply supported bridge gives 176 mm.

With all other modelling parameters being equal, the continuity of the second continuous girder bridge girders causes it to be stiffer and thus have a shorter period than that of its counterpart simply supported girder bridge. Therefore, as presented in *Fig. 5.4*, it is noted that all decks in the continuous girder bridge appear to go through a rigid translation since the bending stiffness of the deck and columns are much greater than the translational stiffness of the bearings.

The deformations for bearings at the start abutment and the left bents that resist seismic loads are reported in the following *Table 5.11*. The overall responses of the steel bearings located at both abutments appear to be very similar regarding the fact that both are expansion bearing.

Table 5.11 Deformation Demand of Support Bearings for bridge-1

Span Name	Station (m)	Bearing Name	Longit. (mm)	Transv. (mm)	Vertical (mm)	R1 (Deg)	R2 (Deg)	R2 (Deg)
Start Abutment	0	1	15.72	0.0635	0.1526	5.43E-04	7.29E-05	1.17E-11
Start Abutment	0	2	15.72	0.0635	0.1522	5.51E-04	3.58E-05	1.29E-11
Start Abutment	0	3	15.72	0.0635	0.1522	5.51E-04	3.58E-05	1.28E-11
Start Abutment	0	4	15.72	0.0635	0.1526	5.42E-04	7.28E-05	1.19E-11
Span1	25.25	1	11.254	8.22E-10	1.32E-09	7.16E-05	0.025	2.71E-04
Span1	25.25	1	1.254	4.79E-09	8.91E-10	6.34E-05	0.025	3.99E-04
Span1	25.25	2	11.254	4.79E-09	8.91E-10	6.34E-05	0.025	4.01E-04
Span1	25.25	2	11.254	8.20E-10	1.32E-09	7.18E-05	0.025	2.70E-04
Span2	50.5	3	11.254	7.17E-10	1.25E-09	1.01E-05	0.025	2.23E-04
Span2	50.5	3	11.254	3.96E-09	8.58E-10	1.58E-05	0.025	3.29E-04
Span2	50.5	4	11.254	3.96E-09	8.58E-10	1.59E-05	0.025	3.30E-04
Span2	50.5	4	11.254	7.16E-10	1.25E-09	1.01E-05	0.025	2.23E-04

Unlike the simply supported case, relatively small deflection of bearings in continuous bridge was resulting with higher seismic demand at the bottom of piers. The bearing deformation under the governing longitudinal loading of the second bridge is about 11 mm. Hence the bearing deformations in the transverse as well in the longitudinal directions are insignificant which may contribute to the increased demand placed on the columns. Therefore, the bearings are not flexible and they do transfer significant forces to the piers and place significant demand on the bridge piers in both the longitudinal and the transverse directions.

As presented on *Table 5.12*, column responses are given in terms of lateral displacements, moment and force demand developed at the foot of circular piers. The axial forces demand for the continuous bridge piers are as high as twice of the simply supported piers.

Table 5.12 Force Demand of the Second Continuous Girder Bridge Piers

Design Request	Bridge Object Name	Station (m)	Column (No)	Location	P (KN)	V2 (KN)	V3 (KN)	M2 (KN-m)	M3 (KN-m)
DReq1	Br-2 Bent-1	25.25	1	Bottom	3684.20	991.98	1581.65	1869.34	2685.58
DReq1	Br-2 Bent-1	25.25	2	Bottom	3684.20	991.98	1581.64	1869.34	2685.58
DReq1	Br-2 Bent-2	50.50	1	Bottom	4388.87	1027.56	1928.28	2280.75	2770.06
DReq1	Br-2 Bent-2	50.50	2	Bottom	4388.87	1027.56	1928.28	2280.75	2770.06
DReq1	Br-2 Bent-3	75.75	1	Bottom	4388.87	1027.56	1928.28	2280.75	2685.58
DReq1	Br-2 Bent-3	75.75	2	Bottom	4388.87	1027.56	1928.28	2280.75	2770.06
DReq1	Br-2 Bent-4	101.00	1	Bottom	3684.20	991.98	1581.65	1869.34	2770.06
DReq1	Br-2 Bent-4	101.00	2	Bottom	3684.20	991.98	1581.64	1869.34	2685.58

One could also note that, higher demand of shear force and moment is resulted in longitudinal direction than that of the transversal direction, while the transversal loading demand is only about seventy percent of the longitudinal; indicating that as far as the columns are concerned longitudinal loading dominates the response.

5.3.2 Effect of Higher Modes in Seismic Response of the Continuous Bridges

This study also has an objective to find out the contribution of higher modes that are considered in the determination the seismic responses of the bridges. The limitations of modal contribution in the computation dynamic responses is based on the modal mass participation factors. The modes to be considered are selected in such a way that the cumulative mass participation should attain 80 % or above of the entire bridge mass. In order to achieve this, the first fifteen modal mass participations of the subject bridges were presented in *Table 5.7* and *Table 5.10.*

Hence from the modal analysis it is found that, for longitudinal loading both bridges show the contribution of only a single mode is sufficient to obtain the seismic responses. Whereas, for transverse loading both bridges also show the domination of a single mode at their third mode.

From the modal mass participation factors one could also note that the fundamental mode of both continuous girder bridges is a longitudinal mode giving modal mass participation of 98%. However, the bridges show the domination of only the first mode in longitudinal loading and the a single mode at the third mode for transversal loading giving mass participation of 99% for the first bridge and 97% for the second bridge. Hence these contributing modes should be considered to determine the dominant dynamic response of the continuous bridges.

Therefore, both continuous bridges show the contribution of only a single mode in both orthogonal loading scenarios. Hence it is also found that only the first mode could be sufficient to simulate the seismic responses of the continuous girder bridges.

5.4 Comparison of Simply Supported and Continuous Bridge Responses

A better understanding of the seismic responses and comparison of these bridge types is obtained by looking at their peak component responses for the entire ground motion suite introduced previously. From the above tables, *Table 5.1* through *Table 5.12,* shows the peak displacement and force demand responses for the entire bridge. *Figure 5.1* through *Figure 5.4,* shows the fundamental mode shapes and the deformation behaviours of the continuous and the simply supported bridges.

From general observation of modal response of the bridges, it could be noted that the response pattern of the simply supported and the continuous bridges become similar. The lowest fundamental mode shapes respond in such a way that the deck slabs goes through a translational response. This implies the fundamental modes of both bridges showed significant lateral deformation rather than bending deformations. From this one could suggest that the bracing systems or the stiffeners should be detailed appropriately at the design stage of both bridge types.

Consequently, the continuity of deck slabs over the supports improves vertical capacity and that could shift seismic demand from one component to another component. In particular, less deformation demand is required of the middle span bearings but more demand is required of the columns as a result. The bearings in the simply supported girder bridges appear to take the majority of the demand in the form of deformations. Thus it is helpful to realize that while attempting to improve the seismic performance of the bridge by using continuous spans one may, increase seismic vulnerability of some components in the design phase, specially the concrete piers.

Furthermore, the multi span simply supported bridge components experience their largest demands when the bridge is loaded in the Transversal direction. However, the bearing deformations under transversal loading are consistently larger than those of the longitudinal direction. Therefore, when assessing the seismic response of this type of bridge it may not be appropriate to only consider a two dimensional model. It is recommended that both orthogonal directions be considered for this bridge type to enable assessment of the peak demands placed on all components. The continuous nature of the multi span continuous girder bridge consistently causes the longitudinal direction to dominate the bridge response. Therefore, one may find it appropriate for a similar bridge type to use a two dimensional longitudinal model and from that make some valid inferences.

From the general modal analysis result it could be seen that the effect of higher modes in seismic response of continuous bridges become insignificant, whereas response of higher modes for simply supported bridges have significant contribution. Hence to predict seismic behaviour it is recommended to use the first fundamental mode for the continuous bridges and all the lowest contributing modes for the simply supported bridges.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This paper presents the results of a seismic response evaluation for two simply supported bridges which are typical to the Ethiopian bridge construction practice and their counterpart two continuous girder bridges. A ground acceleration coefficient, specifically for the Afar area which approximates a 0.1g ground acceleration hazard level, is used to evaluate and compare the seismic responses of these bridges. The bridges are evaluated to understand typical seismic responses and highlight vulnerable components to evaluate the responses under both principal directions, longitudinal and transverse excitation. These loadings are directionally combined with a scale factor of 30% to provide two governing load cases. The analyses in this study focus on two typical three span and five span bridge configurations subjected to horizontal ground motions. It is recognized that the findings from this scenario may vary as the bridge configuration, number of spans, span lengths and column height changes.

From the analysis results it is observed that, the response pattern of the simply supported and the continuous brides is similar. The lowest fundamental mode shapes respond in such a way that the deck slabs goes through a translational response. This implies the fundamental modes of both bridges showed significant lateral deformation rather than vertical and bending deformations. Hence one could suggest that the bracing systems or the stiffeners should be detailed appropriately at the design stage.

Though both bridges respond in a similar fashion, the mass participations of critical modes in simply supported bridges are smaller than that of the continuous girder bridges. This is resulted from the middle span of the simply supported bridge goes through a large translation while the end spans are limited with insignificant response. But in the continuous girder bridges, all decks appear to go through a rigid translation since the bending stiffness of the deck much greater than the translational stiffness of the bearings.

Generally this study gives support to the issue that all critical bridge components should be considered when performing a seismic assessment. An insignificant displacement demand in support bearings of the continuous bridges cause the load to be shifted to the columns causing an increase in demand of force. The axial forces demand for the continuous bridge piers are as high as twice of the simply supported piers. This shows that the continuous nature of the deck girders may tend to increase the demand placed on the bents.

Therefore, one would also note that the demand on the columns would be greater for the continuous girder bridges than for the simply supported girder bridges. This is because of the continuity diaphragm in continuous bridges would enable load to be shifted from the superstructure to the substructure. Hence, with guidance from the seismic retrofitting manual for highway bridges, ductility levels should be checked in accordance to provide seismic detailing of piers. Unfortunately, due to time constraints, in this study these ductility levels are not investigated.

The study has also presented the effect of higher modes in the determination the seismic responses of the bridges. The modal response spectrum analysis results showed that the effect of higher modes in seismic response of continuous bridges is insignificant, whereas the continuous bridges showed the contribution of only a single mode in both loading scenarios. Hence to predict seismic behaviour it is recommended to use the first fundamental mode for the continuous bridges and the contributing lowest modes for the simply supported bridges.

Furthermore, for the simply supported bridges the response spectrum load cases result a quite similar response in the longitudinal and transversal loading. This is resulted from the discontinuity of bridge deck provides similar lateral stiffness of the bridge in the two principal directions. Whereas for the continuous girder bridges, the longitudinal response becomes quite larger than that of the transversal response.

Therefore, it could be suggested that, three dimensional modelling should be employed as well as both longitudinal and transverse loading are significant and should be considered when performing seismic analyses of simply supported bridges. Similarly for the continuous bridges, it could be recommended to use a two dimensional longitudinal model provides an adequate result.

6.2 Recommendations for Further Study

As it was presented, this study is aimed at the seismic assessment of selected multi span bridges subjected to horizontal earthquake excitation. The analysis is carried out using response spectrum tool which is limited to a linear elastic analysis. Hence the adequacy and relevance of utilizing non linear time history analysis and pushover analysis for such bridge types should be studied. Consequently, the findings from this study may vary when the bridge configuration is changed. Among such configurations, the effect of change in number of span, span length, column height, bridge orientation or skewness and provision of cross bracings in seismic response are suggested for future investigation.

Even if the consideration of vertical seismic excitation is not common in some design codes, as well as in this study, in high seismic regions it should be considered. Thus it is recommended that the modelling of bridges for lateral and vertical loading might require further investigation. Furthermore, this research paper presented the displacement and force demand of the selected concrete bridge piers. This demand analysis was utilised for only in the seismic assessment the different structural elements of the bridges. Consequently, it is suggested that the ductility demand of the bridge piers' should be evaluated in order to provide appropriate seismic detailing of reinforcements at critical sections.

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