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**ASSESSING THE SEISMIC VULNERABILITY
AND REFURBISHING FOR REINFORCED
CONCRETE BRIDGE**

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Master's Thesis

Supervisor

Prof. Dr. Tuncer ÇELİK

Istanbul, 2023

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The thesis titled ASSESSING THE SEISMIC VULNERABILITY AND REFURBISHING FOR REINFORCED CONCRETE BRIDGE prepared by AYA HAQI ISMAEL ISMAEL and submitted on 00/00/2023 has been **accepted unanimously** for the degree Master of Science in Civil Engineering.

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I hereby declare that all information/data presented in this graduation project has been obtained in full accordance with academic rules and ethical conduct. I also declare all unoriginal materials and conclusions have been cited in the text and all references mentioned in the Reference List have been cited in the text, and vice versa as required by the abovementioned rules and conduct.

Aya Haqi Ismael ISMAEL

Signature

DEDICATION

With love and affection, I dedicate my thesis:

To my beloved mother, whose pure soul shines on, even in her absence. Losing her has been one of the hardest trials of my life, but her unwavering love and tenderness continue to guide me every day. She was the best giver, always putting others before herself, and her pure soul is a constant inspiration to me. Words cannot express the depth of my love for her, nor the ache in my heart from missing the warmth of her applause. She was not only my mother, but also my mentor and my best friend. I dedicate this thesis to her, with the hope that it may be a small tribute to the immeasurable impact she had on my life. Her kindness will never be forgotten, and her memory will forever be cherished.

To my beloved father, whose name I carry with pride and whose unwavering support has been the source of my happiness and success. I am forever grateful for everything you have done for me.

To my beloved siblings, who have been a guiding light and a source of unwavering support throughout my academic journey. Without your constant encouragement and assistance, this accomplishment would not have been possible. I dedicate this thesis to you with profound gratitude and love.

To my dear friends, who have been my unwavering support system throughout this journey, I dedicate this thesis with utmost gratitude and appreciation for your constant encouragement and love. Your presence in my life has made all the difference.

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ABSTRACT

ASSESSING THE SEISMIC VULNERABILITY AND REFURBISHING FOR REINFORCED CONCRETE BRIDGE

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This research was aim to evaluate seismic susceptibility and rehabilitation of the reinforced concrete bridge. The seismic analysis and seismic response of the bridge under traffic load were analyzed in CSI Bridge 20 according to the AASTHO LRFD and HL-93 standards. A specific bridge of 60 meters in length and 15.5 meters in width was considered for analysis. The bridge was a concrete tee-beam bridge situated in Russia and had four lanes with piles in the foundation measuring 10 meters tall. It was discovered that 3520.03 KN-M was the maximum horizontal axis moment under load combo 1, and 4726.90 was the maximum bridge section stress. The maximum vertical displacement on the bridge span was 1.197E-03 metres, and the largest longitudinal displacement was 8.196E-04 metres. Additionally, Pushover analysis using the ASHTO LRFD 2012 code with built-in load cases determined the bridge's seismic design response. Bridge pier transverse and longitudinal bents were examined. The highest displacement was 42.27mm in pushover curve 7 under load case PO TR4 for bent 4 transversely. In Pushover curve 8, load case PO LG4 for bent 4 in longitudinal direction had 57.25mm displacement. Moreover, the Modal analysis examined how earthquakes affect bridge observation frequencies and times. Six deformed shapes' periods and frequencies were examined first. Mode 5's deformed shape responds well to the bridge's elastic property with $T=$

Keywords: Seismic Vulnerability, Refurbishing, Reinforced Concrete Bridge, Load Resistance Factor Design, Reinforced Cement Concrete.



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ABBREVIATIONS

DL	:	Dead load
L.L	:	Live load
RC	:	Reinforced Concrete
FEM	:	Finite Element Modelling
LRFD	:	Load Resistance Factor Design
RCC	:	Reinforced Cement Concrete

LIST OF SYMBOLS

μ : Electron mobility

Λ : Wavelength

Ω : Angular frequency



1. INTRODUCTION

Concrete structures may be damaged for a variety of causes, including traffic evolution and road accidents, environmental effects, or seismic actions. In areas with a lot of earthquakes, it's likely that buildings will be hit by a lot of quakes over the course of their lives. So, with each earthquake, the level of reliability of structures may drop compared to how they were designed [1]. In order to address these issues, it is required to evaluate the seismic susceptibility and rehabilitation of reinforced concrete bridges.

Bridges are an essential component of the road network; yet, they are susceptible to damage from natural disasters such as earthquakes and hurricanes, as well as from environmental factors like as erosion and collisions between ships [2]. In the past, when people constructed bridges, they would take into consideration the consequences of various dangers in a sequential manner, without giving any thought to how these dangers may interact with one another [3]. Over the course of the previous several years, a number of bridges have collapsed as a direct result of repeated or successive natural catastrophes. This demonstrates that standard techniques of bridge design are not enough to satisfy the requirements of bridge constructions when subjected to many hazards [4]. When constructing structures for scenarios that include numerous risks, you have to take into consideration the unpredictability that exists between the structure's capacity and its demand, in addition to striking a balance between the structure's dependability and its cost.

The deterioration of the bridges is caused by a combination of natural disasters, increased traffic, and unfavourable environmental conditions. Elnashai et al. indicate that the ductility demand on a bridge during many earthquakes is several times more than during a single earthquake. Earthquakes and floods are two natural disasters that happen often and can easily damage or destroy bridges [5]. In order to properly assess the lifetime seismic performance of reinforced concrete bridges in an aggressive environment, it is crucial to take into account both the mechanical damage brought on by diffusion as well as the technique by which aggressive substances like chlorides diffuse. The vulnerability of bridges has increased due to deterioration. Most of what causes a bridge to deteriorate is how it was built, what materials were used, how much it was exposed to the environment, and how old it is. Damage

to concrete bridges can be caused by a number of things, such as changes in traffic patterns, accidents on the road, changing weather conditions, or the way the bridge is loaded.

In addition, the impacts of degradation and damage propagation are compromising the effectiveness of every component in every current kind of bridge. Existing bridges typically require two types of modifications: one to increase user safety by widening the deck and protecting different types of lanes, and the other to mitigate the effects of various types of "mechanical damage [6].

1.1 REINFORCED CONCRETE BRIDGES

Reinforced concrete through-girder bridges consist of a pair of cast-in-place longitudinal girders and a deck slab joined by steel reinforcing bars. These components are held together by the bridge. The components of the bridge made of reinforced concrete are as follows.

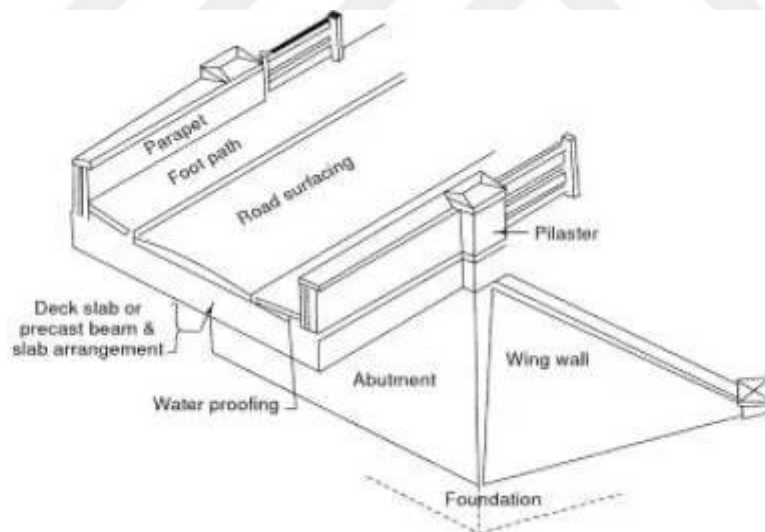


Figure 1.1: Reinforced Concrete Bridge.

1.1.1 Decks

It is common practice to refer to the decking of a bridge as either the road surface or the rail surface. The girders or enormous beams provide support for the decks, and the piers in turn provide support for the decks. The whole configuration is held up by a substantial foundation that mostly consists of piles and a cap system.

1.1.2 Piers

The bearings that are given for load transfer to subsurface soil via the foundation are supported by the piers, which are vertical structures. At various positions along the bridge, these structures act as pillars to hold up the spans of the bridge. In the vast majority of instances, piers are intended to withstand vertical loads by themselves. It is advised that the pier be designed to additionally accommodate lateral loads in regions that are located inside the seismic zone [6]. Concrete is used in the construction of the majority of the piers. In relatively few instances up to this point, steel has been employed for the building of piers. The modern method of pier building involves the use of composite columns, also known as steel columns, that are filled with concrete. The shear mechanism provides the pier, which is a vertical part, with the ability to withstand the forces. The majority of these forces are lateral in nature. The term "bent" refers to piers that are constructed out of numerous columns.

1.1.3 Abutments

The soil is held in place behind a building with the help of abutments, which are vertical structures. The abutments of the bridge are what provide support for both the dead and live loads that come from the bridge superstructure. Additionally, the abutments are subject to lateral forces, which are mostly exerted by the approach embankment.

1.1.4 Bearing

A bearing may be found in the space between the top of the pier and the bridge girder, and it can help sustain the weight. The bearing's principal function is to reduce the amount of impact stress that is delivered to the bridge foundation while also enabling the upper

superstructure to freely move or vibrate. This is accomplished while the bearing fulfils its primary function.

1.1.5 Wing Walls and Returns

The term "wing walls" refers to the structures that are built as extensions of the abutments in order to retain the soil that is present in the approach bank. A natural angle of repose will otherwise exist for this section of the object. These walls are known as retaining walls, and they were built next to the abutments [7]. There is the option of building this wall as a separate structure or as an integrated part of the abutment wall. When being designed, the back of the wall has to take into consideration three different design loads.

1.1.6 Parapets and Handrails

These bridge components are not essential to the bridge's structural integrity but are included out of an abundance of caution. These are located on the uppermost decks of the ship. This will help to prevent the vehicle from falling off the bridge into the body of water below or serve as a means for separating the two lanes of traffic that are currently being travelled on the bridge.

1.1.7 Foundation of Bridges

Foundations are structures that are built to distribute the weight that is placed on them by piers, abutments, and wing walls, and return it across the strata in an equitable manner. The foundation that is supplied for bridge construction is very deep in order to either prevent the bridge from being undermined by scouring caused by the passage of water or lessen the likelihood of undermining occurring.

1.2 LOADS ON BRIDGES

Bridges sustain a variety of loads that are described below:

1.2.1 Dead Load

The structural system consists of the deck slab, primary members (beams or girders), secondary members (bracing, connecting plates, etc.), stiffeners, diaphragms, the floor beam,

and the cross frame. Dead loads will be applied to the superstructure after the deck has dried out and has begun cooperating with the main components to withstand weights. The median would be included in this group, along with the surface,

1.2.2 Live Loads

A bridge is said to be supporting "temporary loads" when it is really only doing so for a limited amount of time. The most important loading condition for temporary work is known as live loads, whereas the most important loading condition for permanent work is known as dead loads.

1.2.3 Earthquake Loads

Earthquake loading is caused by the interaction of natural forces, often known as seismic forces, and the geographic location of the bridge. The construction of every bridge must prioritise the protection of human life while also adhering to the criteria established by the Maximum Considered Earthquake (MCE). It is not inconceivable that the owner of the bridge might decide to implement stricter performance criteria in the event that doing so would facilitate post-earthquake access to emergency facilities or in the event that the rapid restoration of service would have a significant detrimental effect on the economy. In regions where earthquakes are common, seismic forces represent a substantial loading component that typically dictates how bridges should be designed. The magnitude of the seismic activity is proportional to the acceleration coefficient measured at the bridge's position. If the acceleration coefficient of a bridge is more than, it is considered to be located in an area with substantial seismic activity (0.19). The seismic performance category of the bridge is determined by using this coefficient in conjunction with whether or not the bridge is considered critical to the transportation network (SPC). Single-mode spectral analysis, which is based on the SPC and the number of spans, is by far the most common method for conducting an analysis to calculate the loading that the bridge experiences as a result of seismic forces. This is because this method provides the most accurate results.

1.3 SEISMIC VULNERABILITY OF BRIDGES

Bridge failures may be attributed to a wide variety of factors, which can be challenging to classify. However, earthquakes are responsible for the majority of recent examples of damage and collapse across the globe. Bridge structures play an important part in the operation of transportation networks. The destruction caused by earthquakes to these buildings will result in tremendous economic loss. The deck, bearings, piers, and abutments of the bridge all sustained varying degrees and kinds of damage as a result of seismic activity, which in turn had an impact on the bridge system and the activities that took place on it. The dynamics of moving vehicles are another factor that contributes to the expansion of earthquakes in bridges [8].

1.4 PROBLEM STATEMENT

The combination of natural disasters, increasing traffic, and adverse environmental factors is accelerating the deterioration of bridges. Bridges are vulnerable in the event of a moderate to high-intensity seismic event, posing the risk of deterioration and collapse. One of the most important earthquakes was the San Fernando earthquake of 1971, which showed that bridges built before that time were not built to withstand earthquakes. Also, owners and managers of bridges in all developed countries are always dealing with the problem of fixing up old bridges. In fact, many bridges that are already in use have problems with their size, shape, or function. This is due to the fact that they were built to function at levels that were no longer suitable when traffic conditions and structural safety standards increased. The effectiveness of every component in every bridge typology now in use is being harmed by degradation effects.

1.5 AIM AND OBJECTIVES

The aim of research is to evaluate seismic susceptibility and rehabilitation of the reinforced concrete bridge.

- a. To assess the seismic vulnerability of the bridges at the early design stage.
- b. To assess the performance of different structural elements under the traffic load and seismic event.

- c. To assess the bridges modeling regarding seismic actions at an early stage.

1.6 THESIS ORGANIZATION

This thesis is broken down into five chapters. The first chapter provides an introduction to seismic vulnerability and refurbishing for reinforced concrete.

In the second chapter, there is a detailed review of previous research on how vulnerable reinforced concrete is to earthquakes and how to fix it up. The literature review is organized according to research objectives.

The methodology to conduct research, which includes the parameters that were evaluated, the work technique, and the simulation of the system on the CSI, was described in Chapter 3 of the dissertation.

The findings of the study are discussed in Chapter 4. The simulation on CSI, graphs, and tables are discussed, and the last chapter, which is called the "finale," is the conclusion of the whole research study.

2. LITERATURE REVIEW

2.1 GENERAL

Most of the United States' and Western Europe's civil infrastructure was built in the decades after World War II, namely the 1960s and 1970s, and is now far beyond its useful life expectancy. The deterioration of materials and the increase in volume and weight of traffic add to the stresses caused by ageing infrastructure. Many bridges and viaducts were built prior to the advent of contemporary building norms, which place more strict durability requirements on these essential pieces of transportation infrastructure [9].

Development were evolved in numerous changes to the function and design of various types of bridges. For a certain span and application, the bridge type is supposed to provide the greatest material and construction efficiency. The unexpected low performance of the bridges is due to the adoption of a design philosophy and, as a result, a lack of attention to design detail. Throughout the globe, a number of bridges were created during a time when there were no provisions for seismic loads in bridge regulations. In comparison to structures, the failure of a single structural part or link between bridge components is more likely to result in the collapse of the whole bridge structure. Recent bridge failures and structural damage caused by earthquakes have prompted the development of new standards and retrofit procedures to design and assess the structural vulnerability of bridges[26]. In this chapter seismic vulnerability of bridges, performance of structural elements under the traffic and seismic event, modeling knowledge of bridges is included.

2.2 SEISMIC VULNERABILITY OF THE BRIDGES

Typically, the seismic risk assessment methodologies now used by state DOTs concentrate on the existing status of bridges. However, as a result of environmental degradation, structural performance might deteriorate over time. (Rani Ranade 2021) presents the use of a combined durability-fragility evaluation approach to New York and Washington bridges. For durability assessment, corrosion of steel reinforcement is the primary degradation process examined. The shown computational framework is composed of 3 components: corrosion modelling, earthquake fragility analysis,. Due to corrosion damage, all bridges become increasingly susceptible to seismic activity [10].

Bridges are essential to the transportation network. After a quake, bridges must continue to work. To achieve this objective, a vulnerability assessment must be done. (M C Djemal et al 2019) Create a bridge vulnerability index technique. The most influential characteristics on the seismic behaviour of bridges are identified, and an analytical hierarchy process (AHP)-based seismic vulnerability assessment model is constructed to quantify the impact of each component. To analyse the seismic vulnerability and risk levels of existing bridges, a newly developed approach was used to compute and categorise the vulnerability index for bridges [11].

The level of damage incurred by a highway transportation network's individual components, notably bridges, determines the extent to which an earthquake event affects the functioning of the network. It is straightforward to include the seismic damageability of bridges, as described by fragility curves, into the system of risk analysis of a highway network under the seismic hazard. In particular, the nonlinear static approach of constructing fragility curves for a common kind of California concrete bridge is examined in (S,banrejee et al 2007). Capacity spectrum method (CSM) is used to determine spectral displacement, which is then transformed into rotations at the terminals of the bridge's column supports. Developed fragility curves are compared with those generated by nonlinear time history analysis to validate the accuracy of the present analytical method. The results of both the nonlinear static and time history studies corroborate the consistency of the analytically constructed fragility curves [12].

The results of both small-scale and large-scale analyses of seismic risks point to the need for significant changes to BCP. Bridges in Pakistan were recently destroyed by earthquakes (muhammad Khalid 2021). Authorities should assess the current bridges' carrying capacities. According to the West Pakistan Code of Practice for Highway Bridges, the majority of Pakistan's bridge stock was built with seismic loads of 2%, 4%, and 6% of dead loads in mind during construction. Eight genuine bridges are tested, two from each of the four seismic zones (2A, 2B, 3, and 4). The elasticity of the piers was tested, and static and dynamic evaluations were conducted. It has been determined that the capacity of the piers is limited, and the bridges in zone 2A are typically safer. Bridges in zones 2B, 3, and 4 are susceptible from medium to high water levels. Therefore, it is important to perform a thorough analytical

vulnerability evaluation of bridge stock, with a focus on high-risk areas, and to suggest effective seismic retrofitting solutions [13].

It is advised that reinforced concrete (RC) should be strengthened utilising RC strengthening procedures when reinforced concrete (RC) structures exhibit evidence of material degradation. It entails employing steel fibre reinforced concrete (SFRC) and carbon reinforcement composites (CFC) to strengthen the concrete, injecting epoxy resin of the type SikaDur 52 Injection into the concrete, and adding concrete of the type SikaTop 126. (S Hazarullah et al 2022) describes the development of a set of equations as well as a numerical modelling of the required approach for sparse matrices of a large system of equations. The FEM Ansys application was used in combination with a numerical solver to get around the hardware's lack of big-data memory space. The features of the RC material have been modelled using the composite material mixing rules as if it were a homogeneous material. Numerous effective simulations have been done. The findings allowed for the location of high stress locations, which allowed for the identification of large von-Mises equivalent stresses of strength bound in high load scenarios. Additionally, a graphic representation of the development of displacements through time is possible. The study also discovered that a reliable method for keeping track of a building's structural health is the probabilistic technique's dependability design, which uses the security margin as a stochastic variable [14]

According to the actual level of deterioration of their most susceptible sections, such as bridges, Mariano et al. analyse the challenge of managing, maintaining, and planning for improvements to transportation networks in earthquake-prone locations. The issue of transportation network management, upkeep, and future-planning was investigated. Investigation consists of two parts. The vulnerability of a typical bridge within the network to earthquakes must be established first. To do this, its ability to withstand earthquakes will be evaluated. One possible strategy for accomplishing this objective is the creation of fragility curves that account for the fact that corrosion of reinforcing steel is the principal cause of environmental degradation. After the bridges' fragility curves have been calculated, the next step is to evaluate the transportation network's vulnerability by factoring in how traffic patterns change due to bridge damage. To reduce overall risk, handle the immediate post-earthquake emergency period, and direct rescuers to the afflicted and vital locations,

the findings of this pilot research may be utilised as a first step toward a suitable allocation of economic resources in planning of seismic retrofit interventions. U.S. Department of the Interior's (DOI) United States Geological Survey (USGS) also contributed financially [15].

High volumes of traffic are often manageable on highway bridges, however this does depend on their location. The current state of earthquake prediction means that a typical volume of traffic will still be crossing the bridge when an earthquake occurs. Scour effects, in addition to traffic, may damage bridge piers, which, when combined with seismic stresses, can compromise the bridge's integrity. Some research has been conducted on long-span bridges or in high-seismic zones to examine the combined effects of extreme and service loads, but research on typical short- and medium-span bridges in low- and moderate-seismic zones is few. (Matt, et al 2013) To investigate both typical service loads and exceptional loads acting on bridges of varying span lengths, a generic dynamic simulation approach is introduced. Following an explanation of the methodology, numerical research is conducted to determine the behaviour of a typical highway bridge in a state with hilly terrain during an earthquake. The impacts of varying amounts of foot traffic, earthquakes, and erosion on the bridge's structural integrity are investigated. The objectives of this research are attained by the development of a comprehensive SAP2000 model of the bridge and its supporting structures. To account for traffic volume, researchers examine the dynamic interactions between cars in a simulated stochastic traffic flow. It is feasible to regulate how various bridges react by studying how they function under various combinations of severe and service loads. The effectiveness of the conventional "superposition" approach is likewise investigated, this time under a variety of load conditions. The primary research examined the bridge's performance across a wide range of service and high loads [16].

2.3 PERFORMANCE OF DIFFERENT STRUCTURAL ELEMENTS UNDER THE TRAFFIC LOAD AND SEISMIC EVENT

(M. Kawatani et al. 2007) Determine how a steel plate-girder bridge responds to earthquakes when it is loaded with cars and earthquakes are likely to occur. Under "ground motions," the slip of a heavy dump truck is also looked at. A three-dimensional dynamic response analysis is done to get to the goal of this study. In the analysis, a 12-DOFs model of the vehicle is used, which takes into account swaying and yawing as well as bouncing,

pitching, and rolling. Observations from a numerical analysis show that thinking of heavy vehicles as a dynamic system can reduce the response of bridges to low-frequency ground motion, like the record on sites with moderated soil. On the other hand, ground motions with relatively high frequencies, like those seen on stiff soil sites compared to those on moderate soil sites, tend to make bridges' responses to earthquakes a little stronger. It has also been seen that vehicles that are moving may slip more often than vehicles that aren't moving. Also, during an earthquake, it's easier for cars on viaducts to slide than for cars on roads [17].

Historically, traffic and seismic loads have been evaluated independently when evaluating bridges. Evaluating bridge stability under combined seismic and live loads. In-depth probabilistic analyses are performed to develop fragility curves for the bridge system and pinpoint the critical truck placement that increases its susceptibility to earthquakes. We model the effect of truck load on the seismic fragility of a bridge by generating a fragility surface for the critical truck site, the failure probability of which is dependent on the capacity to regulate the weight of the vehicle and the magnitude of the ground motion. Weigh-in-motion data on vehicle weight distribution and truck flow rate are used with this fragility surface to forecast traffic-informed seismic reliability. The method may be expanded to examine the interaction between earthquake and live loads on a wider range of bridges and hazard scenarios, with the goal of deriving reliability-based load combinations that are in line with current tendencies in bridge design.

Amount of traffic is also a What this means for a bridge's response during a seismic event is unclear, although it is known that live load, in addition to its own weight, has a dynamic influence on how a bridge reacts. Furthermore, most bridge design requirements lack clear guidelines for incorporating live load into seismic design. See what occurs when a car and a bridge collide during an earthquake (Wibowo et al 2012). The research includes both practical experimentation and theoretical analysis. experiments centre around shaking a model of a horizontally curved steel girder bridge carrying a variety of typical vehicles at a scale of 2/5. Preliminary findings reveal that the presence of a live load significantly improved performance under conditions of low amplitude vibrations, but had less of an impact under conditions of higher amplitude shaking. The performance is evaluated using

column displacements, abutment shear pressures, abutment uplift, and concrete spalling [18].

Civil engineers all around the globe are having a harder time estimating how much traffic a road bridge can support due to factors like its age and current condition. For highway bridges, the live load is the most critical kind of stress. However, bridges in seismically active regions often need a supplementary seismic examination and frequently undergo retrofitting to meet quite stringent design regulations. In order to evaluate and repair road bridges that are not covered by current construction rules, (Dominik et al 2022) provides an overview of the most recent procedures (Eurocode). Specifically, two case studies from Croatia illustrate the use of these strategies in that country. The first example demonstrates how a seismic evaluation and retrofitting plans may be derived from a routine examination. The second case study bridge, on the other hand, demonstrates how a rapid evaluation and interim repair may be conducted following a major earthquake. Both spans are located on interstates and were constructed in the 1960s. The piers of the first bridge are V-shaped and built of reinforced concrete. The composite girders of the second, more ancient bridge give it away as an older structure. Placed in the county of Sisak-Moslavina. The bridge sustained significant damage during recent earthquakes in the area and requires immediate inspection and reinforcement to prevent its collapse [19].

Due to the current difficulty in predicting earthquakes, there is a good chance that many cars will still be on a bridge when one occurs. Researchers in the field of seismology have historically disregarded traffic volumes. Existing mode-based bridge-traffic interaction models often neglect the significant nonlinear consequences of earthquakes on short and medium-span bridges. However, standard nonlinear seismic analyses performed using commercial or open-source technologies do not explicitly consider the complex dynamic interactions between moving automobiles and bridges. It is now unable to do nonlinear seismic evaluations of typical short and medium-span bridges, which would take into consideration the interactions between the bridge, moving vehicles, and the earthquake. In this article, a nonlinear seismic study of the bridge-traffic-earthquake system using a hybrid simulation approach is provided. Methodological components comprised a stochastic traffic flow simulator, a mode-based fully-coupled model of the bridge-traffic system, and an

OpenSees-based nonlinear seismic analysis platform (suren chan at el 2019). This article takes a look at a specific kind of bridge—one with a medium or short span—that uses skew and curve to get around challenging crossings and topographical restrictions. The performance of the bridge under seismic loads and the effect of additional traffic are then analysed computationally. The results show that the proposed hybrid technique is able to capture the nonlinear seismic performance of the bridge and the complex dynamic interactions between the bridge and numerous vehicles, allowing for accurate forecasting [20].

2.4 BRIDGES MODELLING REGARDING SEISMIC ACTIONS

(jiang at el 2021) developed a model that is both practical and reliable for use in cable-stayed bridges' stay cables . This model takes into consideration the enormous force changes that may be induced by earthquakes. The model that has been proposed may approximate the nonlinear connection that exists between the equivalent stress and strain that cables experience. In order to do this, it takes into consideration not just how the equivalent modulus of cables changes during an earthquake but also the likelihood that connections would get disconnected. The performance of cables based on the suggested technique is quite similar to the performance of cables based on how cables really behave. After that, the model is used to a case of a cable-stayed bridge that is subjected to seismic activities, and the response of the structure is compared with the conventional technique. The conventional method models the cables with a constant equivalent modulus that is obtained from the dead load condition. The conventional technique does not provide an adequate description of the cable force at low levels of stress and demonstrates changes in the amplitude and phase angle of the structure's reaction over time when compared to the model that has been proposed. In most circumstances, the old technique may result in an underestimate of the bending moment at the base of the tower as well as the longitudinal displacement at the end of the deck, which may result in a seismic design that is less safe. This demonstrates how vital it is to make use of the proposed models when coming up with earthquake-resistant designs for cable-stayed bridges [21].

When bridge girders and decking shift during an earthquake, it compromises both their capacity to function properly and their level of safety. Recent earthquakes of significant

magnitude have shown that, in addition to damage throughout the length of the bridge, pounding on neighbouring bridge girders may produce lateral displacement and rotation of bridge girders, both of which can cause unseating of bridge occupants. For the purpose of simulating this impact, there is an urgent need for the development of 3D models of the dynamic behaviour of whole bridge structures, which must include pounding. (ping at el 202) developed A three-dimensional model which was proposed as a tool for doing precise research on the pounding that occurs between bridge girders. The hammering model that was proposed was put through its tests to see whether it is effective. There has also been discussion on the non-linear modelling of steel raised bridges in three dimensions. The Dynamic Examination of Bridge Systems (DABS) application is a tool that may be used to do an analysis of bridges in a variety of distinct ways. On a three-span steel bridge, seismic assessments are performed for a variety of various scenarios, such as pounding, which is utilised as a case study. The results of these analyses are employed. The calculations show that the model that was suggested for usage with hammering may be implemented successfully. Even if the highest value of the rotating angle is rather low, it is possible to notice a remarkable growth in the girder's rotating angle. According to the findings, the reactions in either the longitudinal or transversal directions are not sensitive to changes in the friction coefficients within the parameters of the calculation scenarios that were selected [22].

Different bridge portfolios and modelling parameters will have different effects on how bridges respond to earthquakes. Most of the time, these features are established before modelling bridges, but choosing them also involves some uncertainty. Estimated seismic demand models are used to assess bridges' dependability, therefore understanding how sensitive these demands are to changes in bridge characteristics is essential. The (Farahanaz 2021) employs analysis of variance and other statistical approaches to determine the impact of various bridge portfolio elements, such as abutment type, on the most crucial of the engineering demand parameters, deck displacement. In addition, the Random Forest ensemble learning approach, a machine learning algorithm, was presented to determine which modelling factors are most crucial when trying to predict seismic requirements. This article provides a framework for analysing concrete box-girder bridges with tall piers, which are often constructed in response to the difficult terrain of the building site, such as in

mountainous or valley areas. It is possible to apply the suggested framework to additional types of bridges. Moreover, past studies have shown that tall pier bridges react differently to earthquakes than conventional bridges. It was shown that bridges with flexible diaphragm abutments were more vulnerable than those with seat abutments.

The number of spans, arrangement of the abutments, and column cross-sectional form are all factors in stiff abutment bridges. In most cases, the direction of ground motion, the configuration of the foundation, and the kind of concrete used in the bridge's superstructure have little to no effect on the bridge's reactions [23].

Bridges are the most important parts of modern transportation networks, while roads are the most important lifelines of today's networks. It is possible for their structural defects to go undetected, which leaves them vulnerable to failure. Analysis of the seismic susceptibility of already-built buildings has received a considerable amount of attention in the aftermath of the recent large earthquakes that caused widespread destruction. The analysis and design of bridges require a significant amount of complication, yet seismic resistance and serviceability are becoming an increasingly important consideration. A significant portion of the world's bridges were built at a time when there were either no seismic safety requirements outlined in the relevant bridge codes or when the criteria included in those codes were inadequate when compared to the requirements outlined in current bridge codes. In addition, a number of India's existing bridges are degrading as a result of the effects of ageing as well as the growing size and volume of vehicle loads. The Box Girder Bridge is now an integral part of bridge networks all over the world. In addition, the Box Girder Bridge is often used for ignoring the heavyweight of other bridge girder networks across long spans. This is possible because of the bridge's box-shaped design. design the modelling that is utilised to study the dynamic response of Box Girder Bridges. This modelling is called CSiBridge version 21.1.0.

The main goal of this research was to assess and look into the seismic output of a 100-meter-long Girder Bridge frame with up to five spans. The amplitude of the peak ground motion and the acceleration of the ground motion both affect how effectively the Box Girder Bridge performs during earthquakes. To properly do this, the nonlinear time history analysis method is employed. Deformed shape, relative acceleration, relative velocity, base shear, base

reaction, shear force, stresses, base moment, torsion, and relative displacement are all indicated by the response results in terms of deformation. Base shear, base reaction, shear force, shear force, stresses, base moment, relative displacement, and base reaction are also indicated [24].

2.5 SUMMARY

This chapter reviews the existing literature about the seismic vulnerability of bridges, performance of bridges under traffic and seismic events, modeling of bridges for the seismic event. There is a shortage of modelling competence in the vulnerability and performance of structural components under traffic and seismic events, despite the fact that multiple model approaches were employed to study the seismic response of bridges in the current literature

3. METHODOLOGY

3.1 INTRODUCTION

Seismic analysis is a form of structural analysis that examines the behaviour of a structure during an earthquake. In areas with an earthquake risk, it is used to plan, evaluate, and repair buildings. In this investigation, the seismic analysis and seismic response of the bridge under traffic load were need to be analysed. In CSI Bridge 2.0, a specific bridge was considered for analysis. This chapter discusses the bridge section details, an introduction to the software, and the analysis details of the modelling steps.

3.2 BRIDGE DETAILS

The bridge that was considered for the analysis is situated in Russia. It is a concrete tee beam bridge. The bridge measures 60 meters in length and 15.5 meters in width. The bridge has four lanes. The pile stands 10 meters tall.

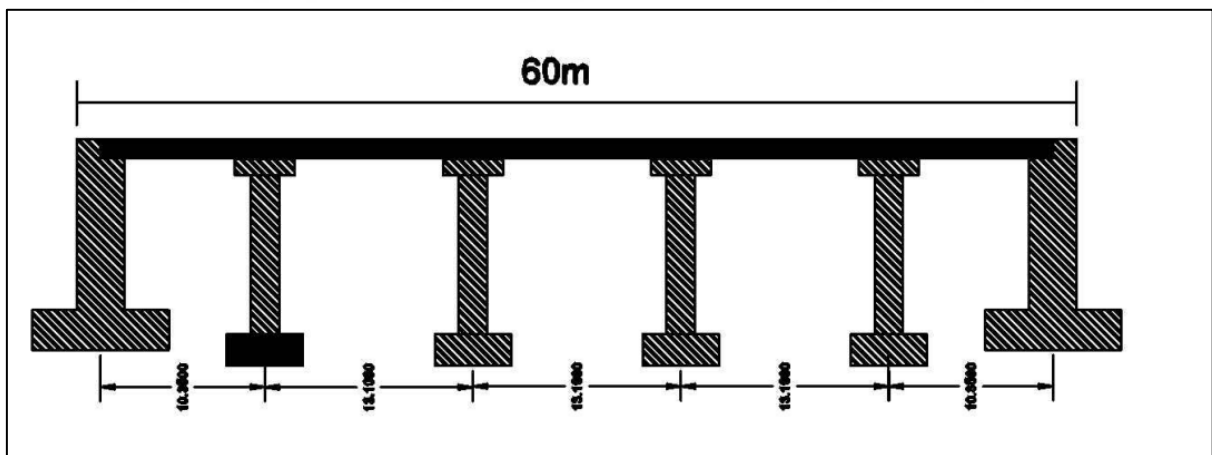


Figure 3.1: Bridge Span Details.

3.3.1 Dynamic Load Analysis

Traveling load analysis simulates the impact of real cars moving on the bridge by using code-specified vehicles or locally produced vehicle models that are described in the programme and operated. At the discretization sites throughout the length and breadth of the lanes, influence surfaces are constructed for the greatest and lowest impact of the loads during analysis. Unit loads known as influence loads are applied at the discretization sites to help build these influence surfaces. The analysis results can be improved to a certain extent by increasing the discretization points because they indicate the longest range permitted between load points, but only to a point where the accuracy is not significantly improved and only disc storage, computational time, and memory are used more.

3.3.2 Linear Analysis

In a linear static analysis, the ratio of stress to strain remain constant and there is a linear relationship between applied forces and displacements. This is relevant in real life to structural issues when the material's linear elastic range is still being affected by stresses. Additionally, we can state that throughout the investigation, the structural characteristics such as stiffness and damping remain constant. Initially, there is no tension in the analysis. Even though it makes use of the stiffness from a prior non-linear analysis, it does not include any loads from any earlier analyses.

3.3.3 A Floating Lane and a Fixed Lane

A floating lane is a kind of lane that may be set up in the software and whose transverse position will be automatically adjusted during analysis to have the maximum effect on each response quantity (load combinations). The road is pulling a live load in the form of a moving vehicle down its side, increasing the strain on the girders. Non-zero and unchanging width characterise this lane.

3.3.4 Non-Linear Analysis/Seismic Analysis

A ratio of stress to strain that varies as a result of a nonlinear connection between the forces that are applied and the displacements that are made is an example of a nonlinear analysis. Nonlinear effects may be caused by a number of different things, including geometrical

nonlinearities, large deformations, and material nonlinearities, such as elasto-plastic materials. When a load is applied, the stiffness matrix undergoes transformations as a consequence of these parameters. On the other hand, when a linear static analysis was performed, the stiffness matrix remained unchanged. In this particular instance, the analysis takes into account all of the loads, deformations, stresses, and other elements that were taken into consideration in the non-linear analysis that came before it.

3.4 FINITE ELEMENT MODELLING

The finite element method, often known as FEM, is a modern numerical approach that is used to analyse finite elements. When the member is discretized (divided down into smaller components) and the reconnecting of these pieces is helped by nodes. [Cause and effect] The whole of the structure, including each individual component, was analysed in order to get the desired results, which may include stresses, strains, and other parameters. The following three stages are included in the software package:

- a. Pre-process
- b. Process
- c. Post-process

During the pre-processing phase, the user is prompted to choose the kind of analysis they want to conduct—either modal or structural static analysis—and materials are defined. Nodes are also created. Nodes bind items together. At this point, the loads are applied, and the boundary conditions are determined.

The computer is responsible for the processing portion of the procedure. During this phase of the process, the computer will go through the boundary value problem and then provide the results to the user. During the step of post-processing, the user is granted access to the results that were produced.

3.4.1 Links

A unique structural behaviour may be modelled thanks to the use of link objects, which connect two joints that are separated by a certain distance from one another.

3.5 NUMERICAL MODELING PROCEDURE

The procedure of numerical modelling is split down into a number of sequential procedures that may be followed with the assistance of the software's wizard guide, instructional links, and manuals. These processes can be accessed via the CSi Bridge programme. Due to the fact that there are two distinct sorts of models, we shall discuss each form of model individually. First, an explanation is given on the creation of the RCC model that is easy to draw. The pre-stressed model, in addition to including information on other properties, also includes information on the arrangement of the tendons (tendon profile coordinates).

3.5.1 Numerical Modeling in Software

The underlying numerical modelling technique used by the programme is the same for both RCC Bridges and Pre-Stressed Girder Bridges. The flow diagram outlines the following main procedures, which are included in both types of bridges:

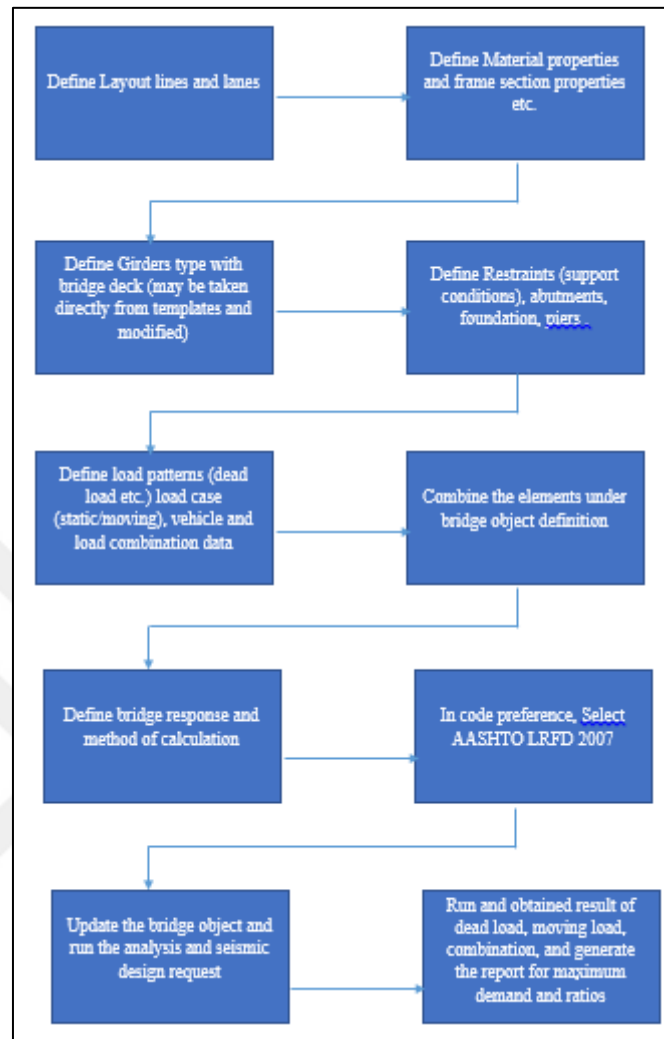


Figure 3.3: General Bridge Construction Methodology.

3.6 DESCRIPTION & MODELLING STEPS

Step 1:

A new model was created in CSI bridge software with units in kilo newton and meters.

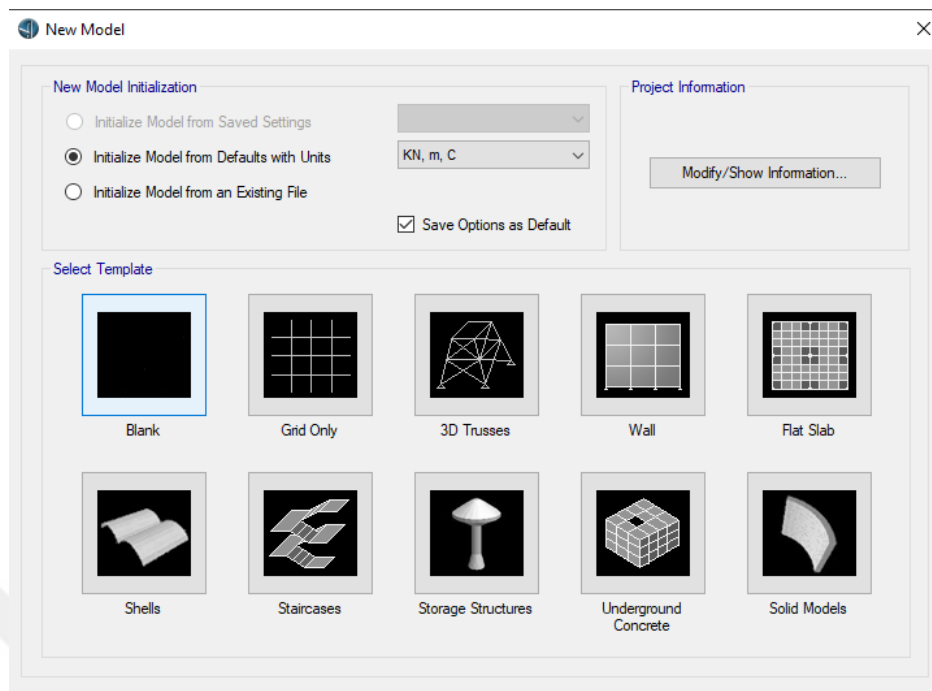


Figure 3.4: The Length of the Bridge Was Allocated as 60m.

Step 2:

The length of the bridge was allocated as 60m.

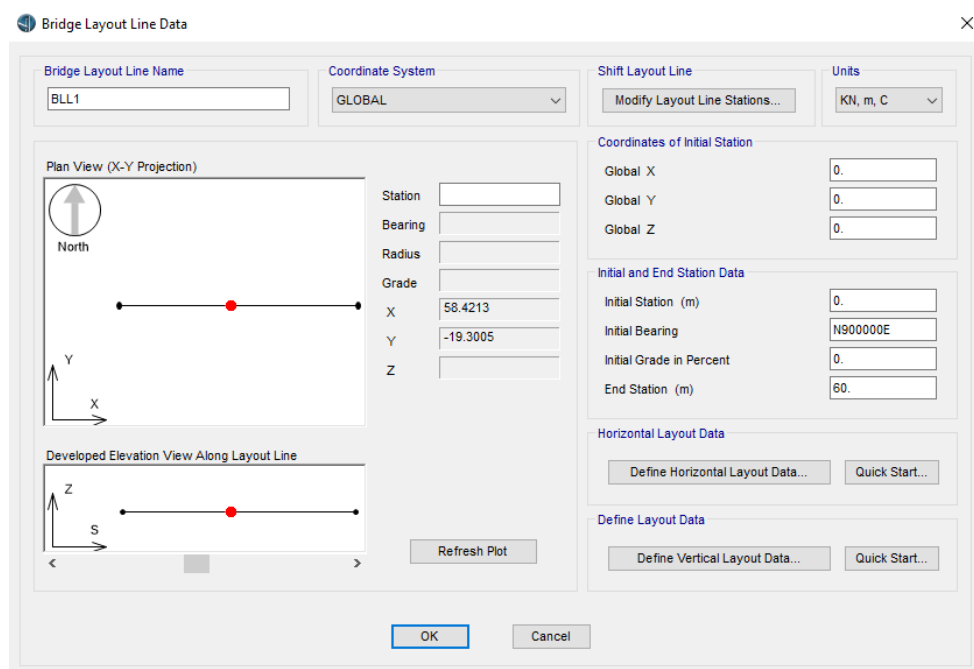


Figure 3.5: Length of Bridge.

Step 3:

Next, 4 fixed lanes were allocated to bridge with the width of 3.53m

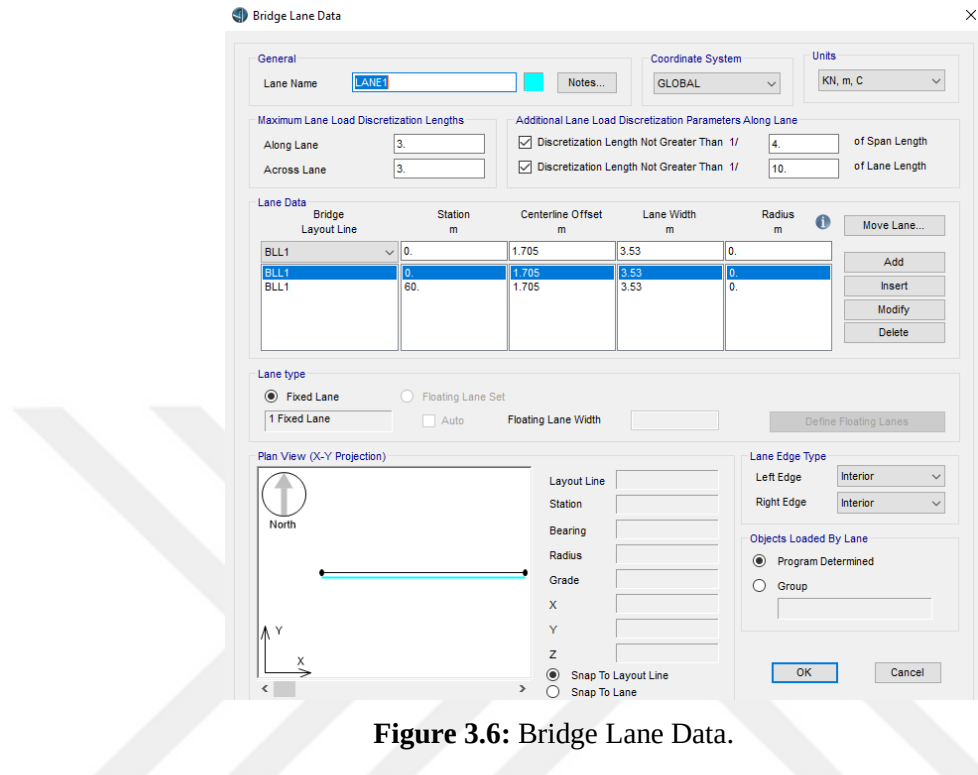


Figure 3.6: Bridge Lane Data.

Step 4:

Following that, material property of 4000 psi was opted. In the frame properties, a rectangular section for abutments was chosen with abutment depth $t_3 = 4$ meters and width $t_2 = 2$ meters. In addition, a rectangular section with the name capbent was chosen with depth $t_3 = 2$ meters and width $t_2 = 1.5$ meters, and a circular section with the name pile diameter = 1.8 meters was chosen.

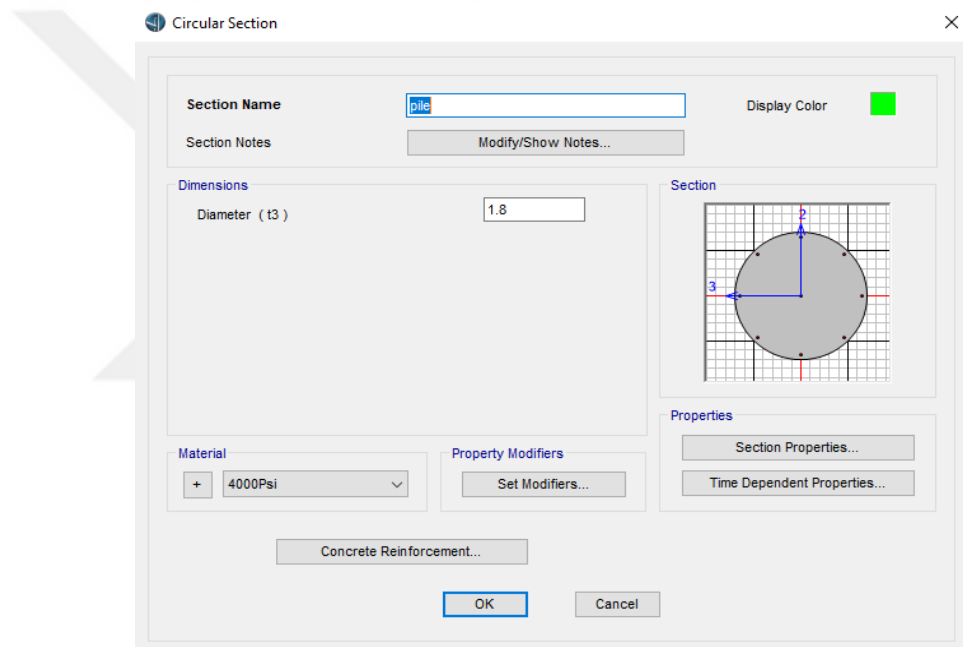
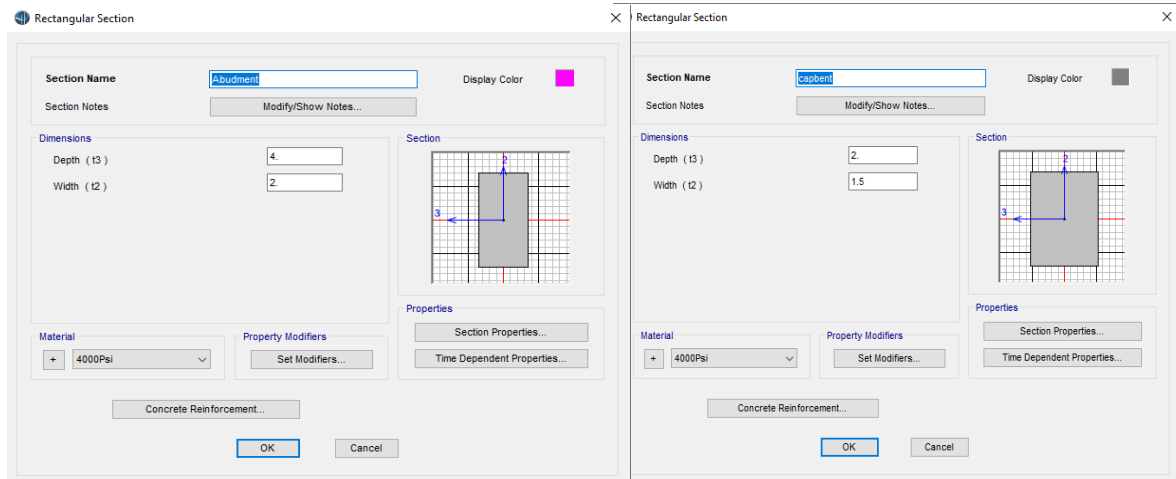


Figure 3.7: Material and Frame Section.

Step 5:

Next, the girder material was select as 4000 psi with width of deck section = 15.5m and depth = 1.2m

Define Bridge Section Data - Concrete Tee Beam

Section Data

Definition

Loads

Item	Value
Exterior Girder	
Exterior Girder Depth Above Flare (L3)	0.7
Exterior Girder Flare Depth (L4)	0.3
Exterior Girder Thickness Above Flare (I3)	0.35
Exterior Girder Thickness Below Flare (t10)	0.35
Interior Girder Data	
Interior Girder Depth Above Flare (L5)	0.7
Interior Girder Flare Depth (L6)	0.3
Interior Girder Thickness Above Flare (t4)	0.35
Interior Girder Thickness Below Flare (t11)	0.35
Left Overhang Data	
Left Overhang Length (L1)	0.25
Left Overhang Outer Thickness (t5)	0.2
Right Overhang Data	
Right Overhang Length (L2)	0.25
Right Overhang Outer Thickness (t6)	0.2
Insertion Point Location	
Offset X From Reference Point To Insertion Point	0.
Offset Y From Reference Point To Insertion Point	0.

Y

X

X: 11.9532

Y: -2.6962

☒ Do Snap

Section is Legal

Show Section Details...

Girder Output

Modify/Show Girder Force Output Locations...

Modify/Show Properties

Units

Materials...

Frame Sects...

KN, m, C

Modify/Show Load Patterns

Load Patterns...

Convert To User Bridge Section

OK

Cancel

27

Step 6:

Next, two bearing one fix another free were allocated

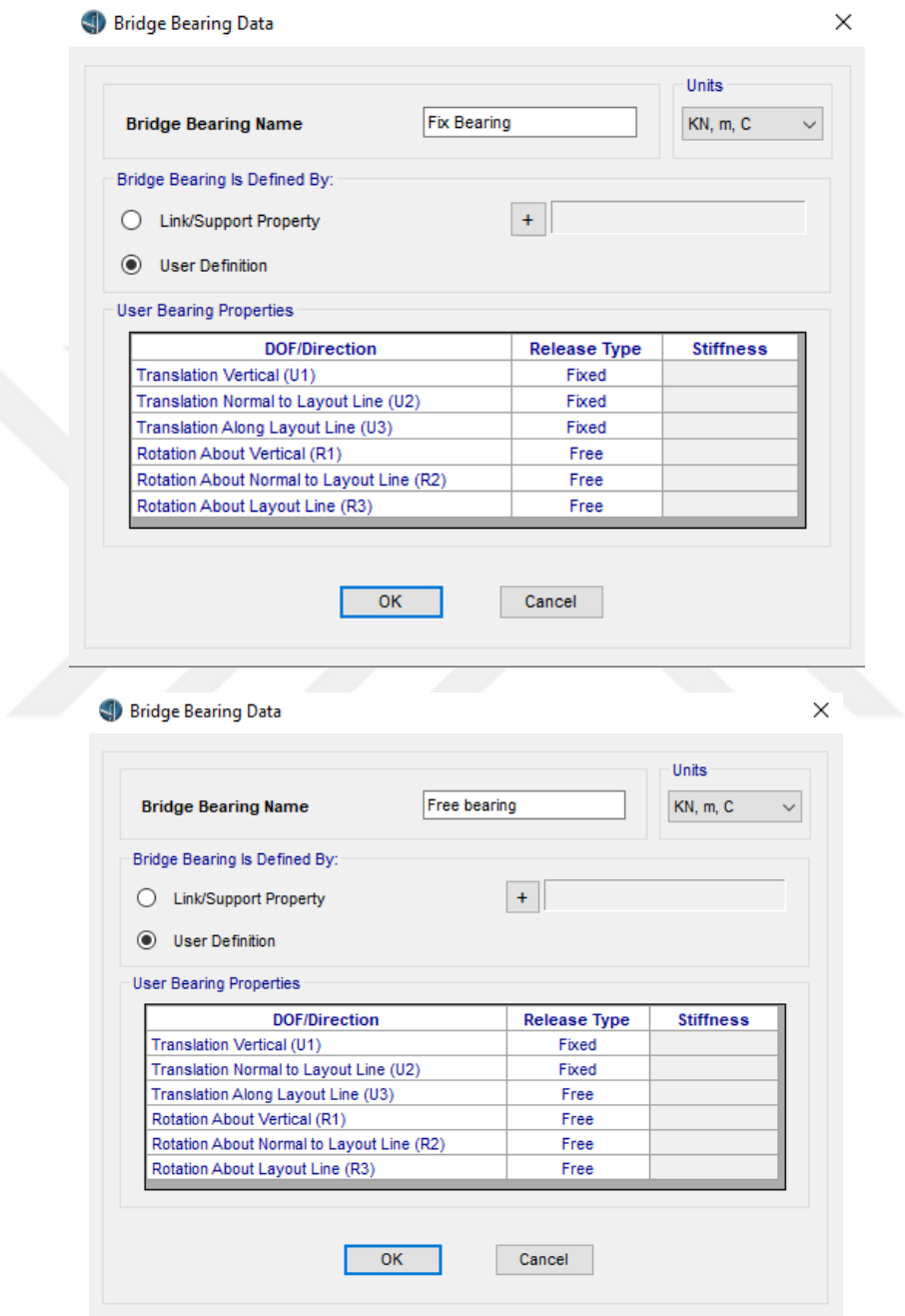


Figure 3.9: Bearing Data.

Step 7

In substructure abutments and cap bent beam length and cap beam length was selected as 14.6m number of columns 4,height of column 10m and distance between columns 1.3m,5.3m,9.3m,13.3m

The image displays two screenshots of software dialog boxes used for defining bridge substructures.

Bridge Bent Data Dialog:

- Bridge Bent Name:** Capbent
- Units:** KN, m, C
- Girder Support Condition:** ☒ Connect to Girder Bottom Only
- Bent Data:**
 - Cap Beam Section:** + capbent
 - Cap Beam Length:** 14.6
 - Number of Columns:** 4
 - Modify/Show Column Data...** button
- Bent Type:** ☒ Single Bearing Line (Continuous Superstructure)
- Buttons:** OK, Cancel

Bridge Abutment Data Dialog:

- Bridge Abutment Name:** Abudment
- Units:** KN, m, C
- Girder Support Condition:** ☒ Connect to Girder Bottom Only
- Substructure Type:** ☒ Continuous Beam (Continuously Supported)
 - Section Property:** + Abudment
 - Beam Length:** 14.6
- Foundation Spring:** ☒ Fixed
- Note:** When substructure type is grade beam, foundation spring property represents a line spring.
- Buttons:** OK, Cancel

Figure 3.10: Butments And Bent.

Bridge Bent Column Data

Bridge Bent Name: Capbent

Modify/Show Properties: Frame Section Properties... Foundation Spring Properties...

Units: KN, m, C

Column Data

Column	Section	Distance	Height	Angle	Foundation Spring
1	pile	1.3	10.	0.	Fixed
2	pile	5.3	10.	0.	Fixed
3	pile	9.3	10.	0.	Fixed
4	pile	13.3	10.	0.	Fixed

Seismic Hinge Data

Column	RH Long	RH Trans	Hinge Prop. Top	Hinge Prop. Bottom
1	1.	1.	Auto	Auto
2	1.	1.	Auto	Auto
3	1.	1.	Auto	Auto
4	1.	1.	Auto	Auto

Moment Releases at Top of Column

Column	R1 Release	R2 Release	R3 Release	R1 Stiffness	R2 Stiffness	R3 Stiffness
1	Fixed	Fixed	Fixed			
2	Fixed	Fixed	Fixed			
3	Fixed	Fixed	Fixed			
4	Fixed	Fixed	Fixed			

OK Cancel

Figure 3.11: Bent Column Data.

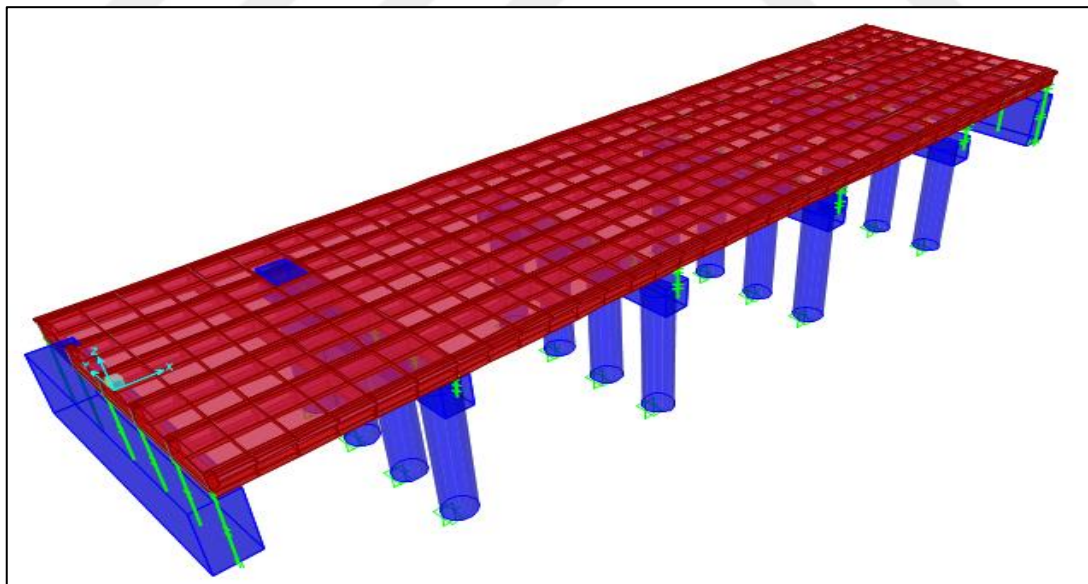


Figure 3.12: Model of Bridge.

3.7 LOAD COMBINATION AND LOAD FACTORS

The load combination were define by the software based on table 3.4.1.1 from ERRATA to LRFD Design load combination and load factors

Load Combination Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time				
										EQ	BL	IC	CT	CV
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I— LL, IM & CE only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— LL, IM & CE only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

Figure 3.13: Load Combination and Load Factors.

3.8 LIVE LOAD

The structure is subject to dead load also with live load in bridge structure is subject vehicle load which considered as live load on bridge live which is use on this bridge model is AASTHO LRFD the type of live load which is considered bridge HL-93 and Moving load lane factor for each lane. Vehicle live load Moving load lane factor are displayed in model,

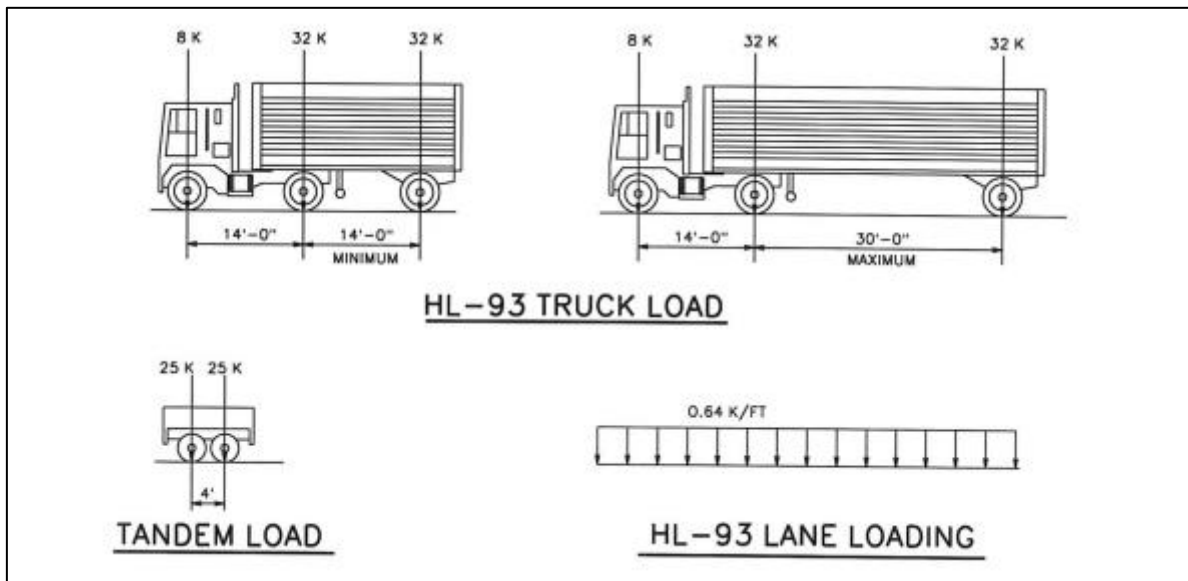


Figure 3.14: HL-93 Loading.

Vehicle Class Data

Vehicle Class Name:

Define Vehicle Class

Vehicle Name	Scale Factor
HL-93K	1.
HL-93K	1.
HL-93M	1.
HL-93S	1.

Add

Modify

Delete

OK

Cancel

Figure 3.15: Vehicle Load.

Load Case Name
Moving load-1

Load Case Type
Moving Load

Stiffness to Use
☒ Zero Initial Conditions - Unstressed State
☐ Stiffness at End of Nonlinear Case

Directional Factors
☒ Vertical
☐ Braking/Acceleration
☐ Centrifugal

Loads Applied

Assign Number	Vehicle Class	Scale Factor	Min Loaded Lanes	Max Loaded Lanes	Lanes Loaded
1	VECL1	1.	0	0	All

MultiLane Scale Factors

Number of Lanes Loaded	Reduction Scale Factor
1	1.2
2	1.
3	0.85
4	0.65

Lanes Loaded for Assignment 1

List of Lane Definitions: [Empty]

Selected Lane Definitions: LANE1, LANE2, LANE3, LANE4

Mass Source
MSSSRC1

Figure 3.16: Moving Load Lane Factor.

3.9 DEAD LOADS:

The structure is subjected to a number of different dead loads, some of which include the self-weight of the girder, the deck slab, the asphalt wearing course, the pedestrian, railing and the sidewalk boarder define in load pattern. Dead loads are displayed in model as:

Define Load Patterns

Load Patterns

Load Pattern Name	Type	Self Weight Multiplier	Auto Lateral Load Pattern
DEAD	Dead	1	
DEAD	Dead	0	
Barrier	Dead	0	
Sidewalk	Dead	0	
Future WS	Wearing Surface	0	
Temperature Pos	Temperature	0	
Temperature Neg	Temperature	0	
Asphalts	Wearing Surface	0	
Railings	Dead Manufacture	0	
Borders	Dead Manufacture	0	
Pedestrains	Pedestrian LL	0	

Click To:
 Add New Load Pattern
 Modify Load Pattern
 Modify Lateral Load Pattern...
 Delete Load Pattern
 Show Load Pattern Notes...

OK Cancel

Figure 3.17: Define Load Patterns.

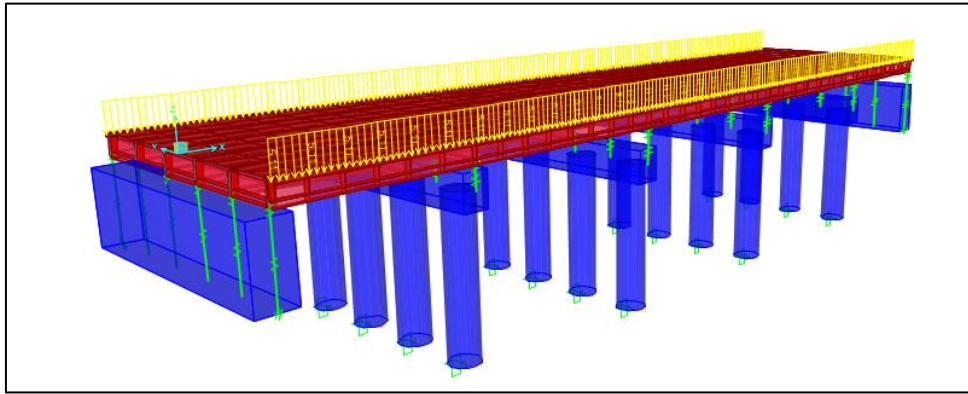


Figure 3.18: Barrier Load.

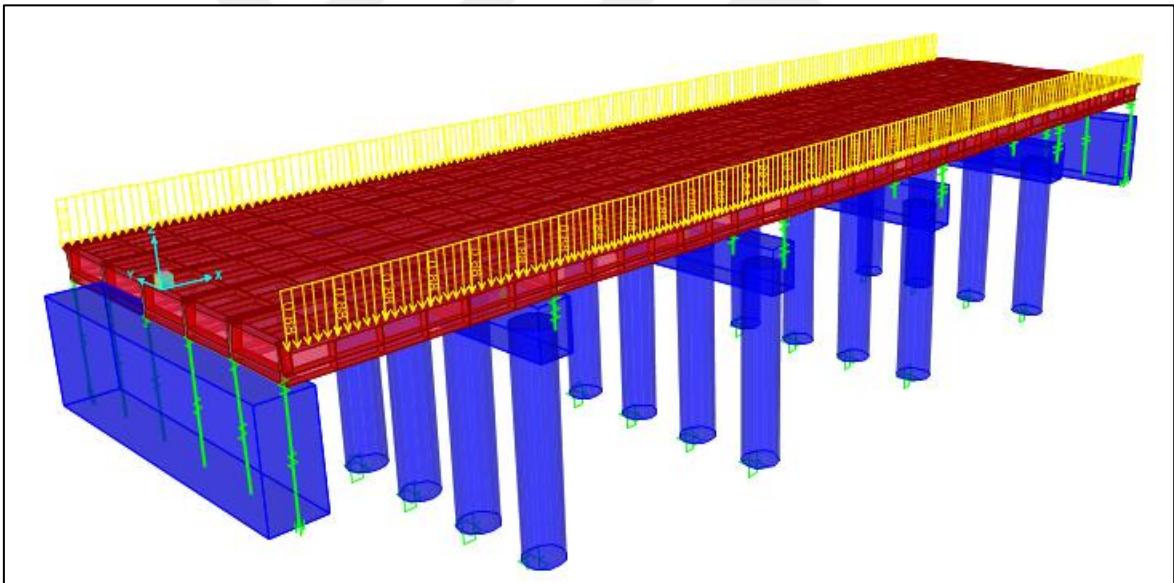


Figure 3.19: Sidewalk Load.

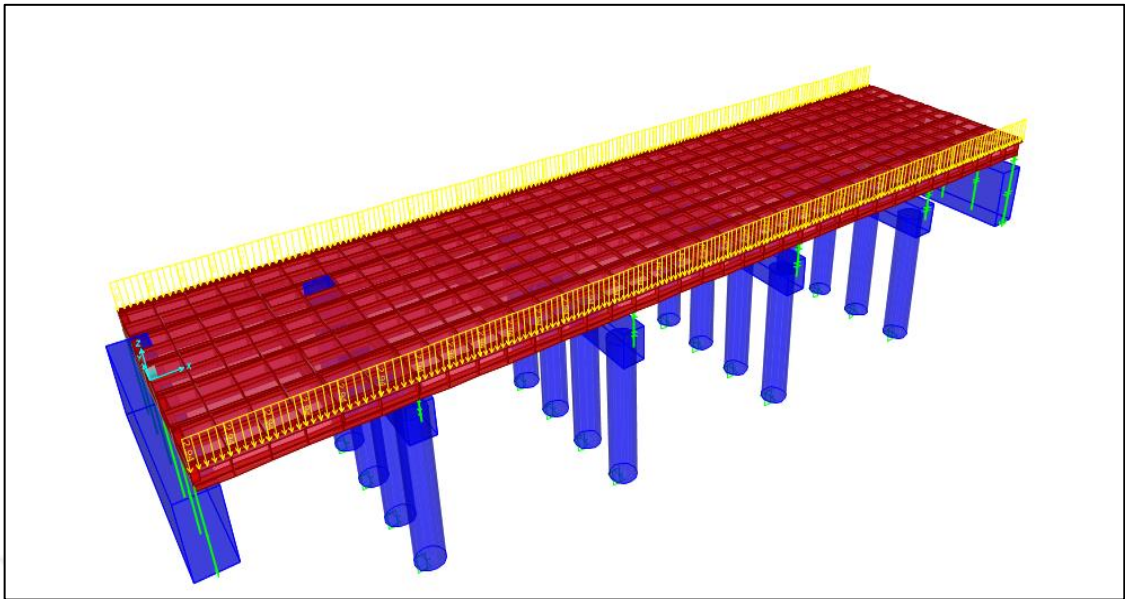


Figure 3.20: Railing Load.

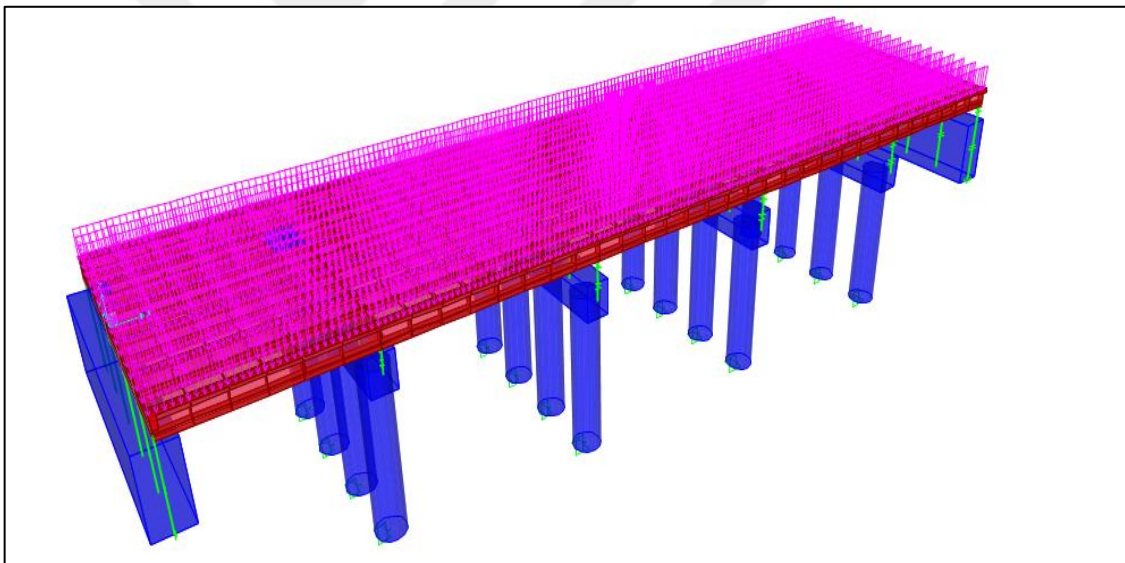


Figure 3.21: Asphalt Load.

4. ANALYSIS AND RESULT

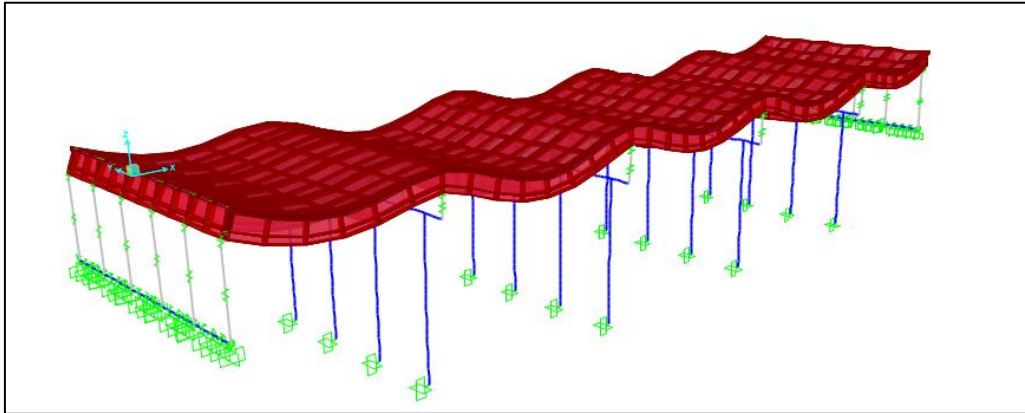


Figure 4.1: Analysis of Bridge Under Different Loads.

4.1 DEAD LOAD RESULT

4.1.1 Result Type Force

In the case of a dead load, result type force, the reinforcement concrete bridge analysis results are shown in. Figure 4.1.1 shows the moment about the horizontal axis for the entire bridge section, which has a length of 60 meters. The maximum moment about the horizontal axis was calculated as 1177.72 KN-M, and the minimum moment about the horizontal axis was -2025.067 KN-M.

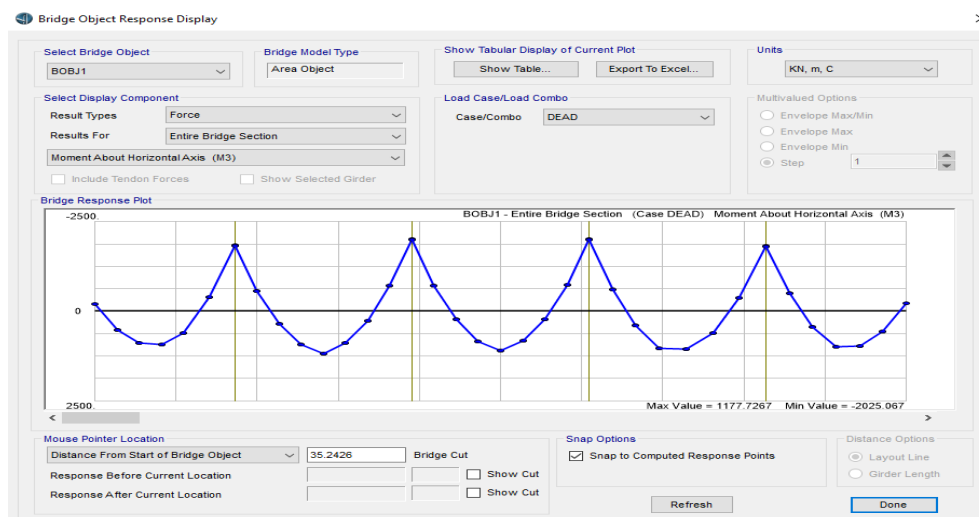


Figure 4.2: Dead Load Moment.

4.1.2 Result Type Stress

The results of the stress analysis performed in the CSI bridge software under the dead load scenario are shown in Figure 4.1.2 for the whole 60-meter bridge section span. Calculated values for the longitudinal stress at the top and bottom center were $1550.70 \text{ KN}/\text{M}^2$ and $-2768.3270 \text{ KN}/\text{M}^2$, respectively.

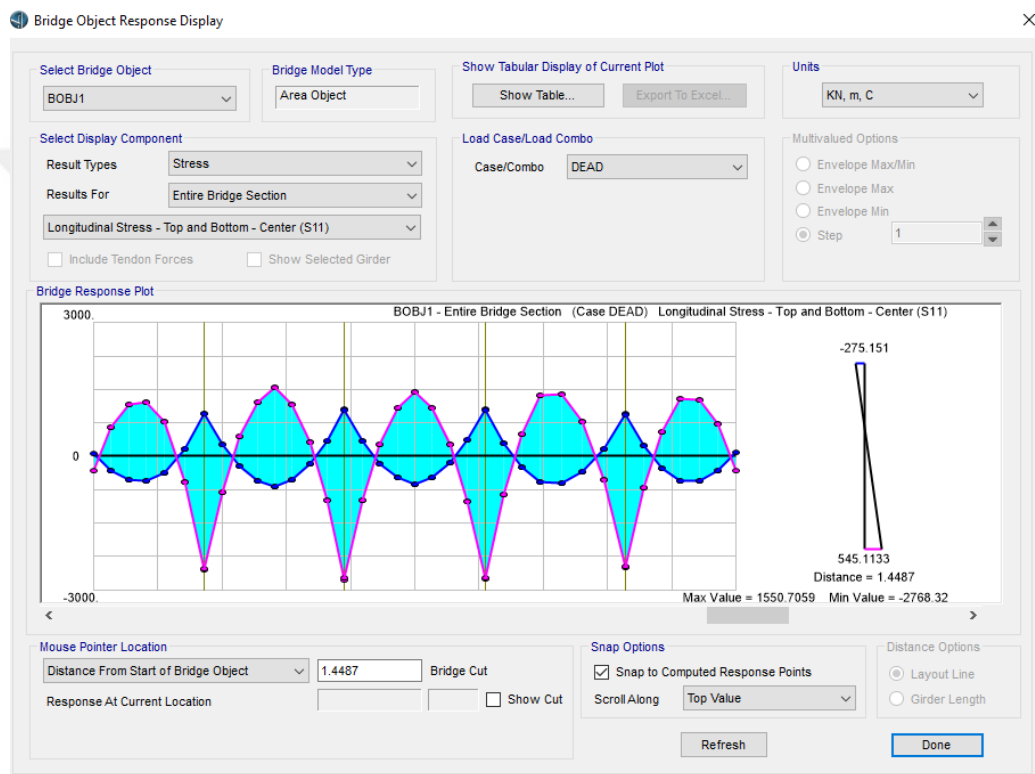


Figure 4.3: Deal Load Stress.

4.1.3 Result Type Displacement

4.1.3.1 Vertical displacement

The reinforcement concrete bridge analysis results are shown in the case of a dead load, result type displacement. Figure 4.1.3 shows the vertical displacement at the entire bridge section, which has a length of 60 meters. The maximum vertical displacement was measured at $-7.591\text{E}-06$ meters and the minimum at $-9.846\text{E}-04$ meters, respectively.

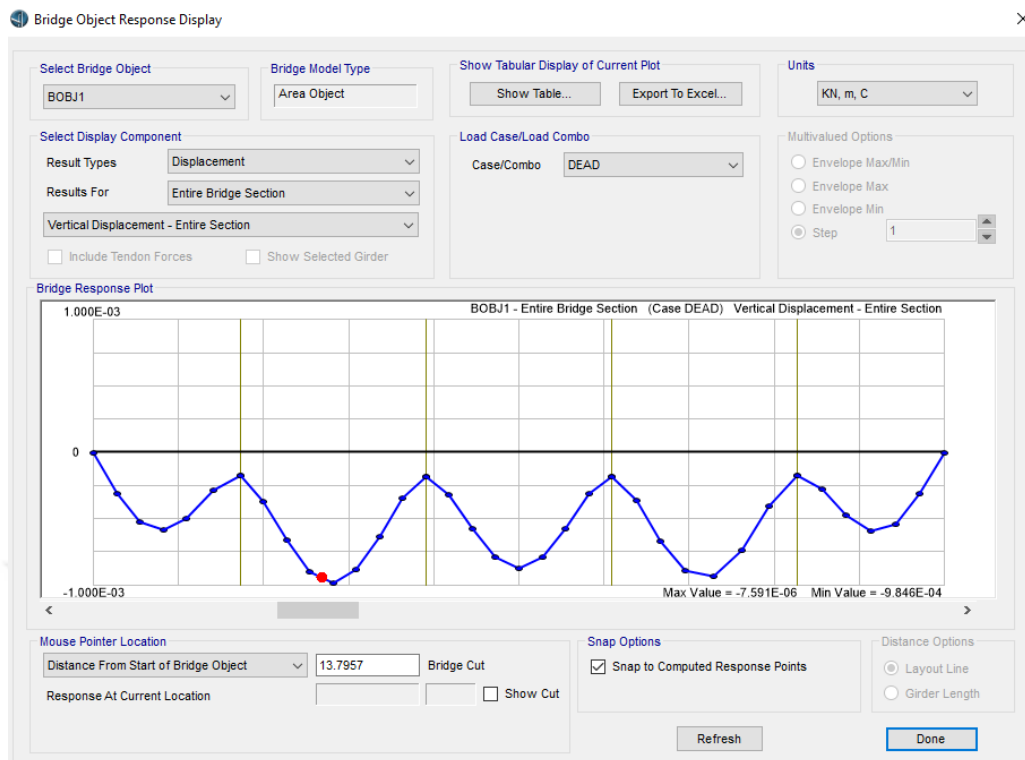


Figure 4.4: Dead Load Vertical Displacement.

4.1.3.2 Longitudinal displacement

The results of the reinforcement concrete bridge analysis for a dead load and result type displacement are shown. Figure 4.1.3.2 shows the longitudinal displacement at the entire bridge section, which has a length of 60 meters. The largest longitudinal displacement was $2.323\text{E-}04\text{E-}06$ meters, and the smallest was $5.746\text{E-}05$ meters.

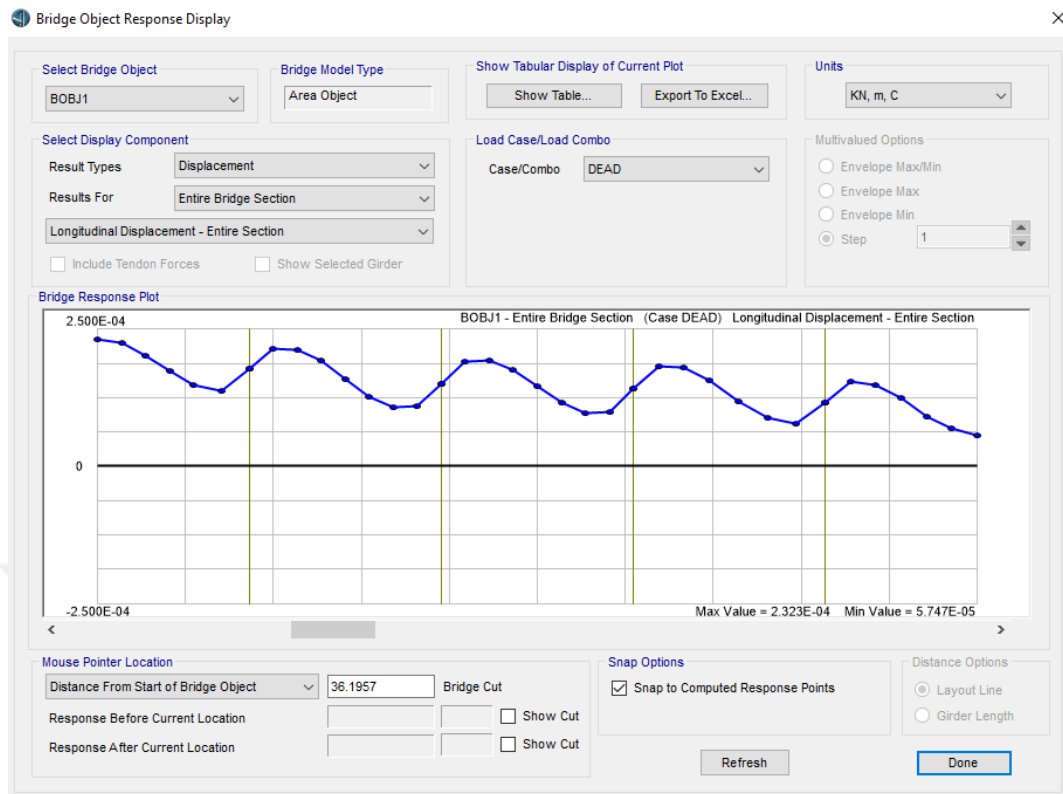


Figure 4.5: Dead Load Longitudinal Displacement.

4.2 MOVING LOAD RESULTS

4.2.1 Result Type Force

In the case of a moving load, result type force, the reinforcement concrete bridge analysis results are shown in. Figure 4.2.1 shows the moment about the horizontal axis for the entire bridge section, which has a length of 60 meters. The maximum moment about the horizontal axis was calculated as 1821.25 KN-M, and the minimum moment about the horizontal axis was -1578.32 KN-M.

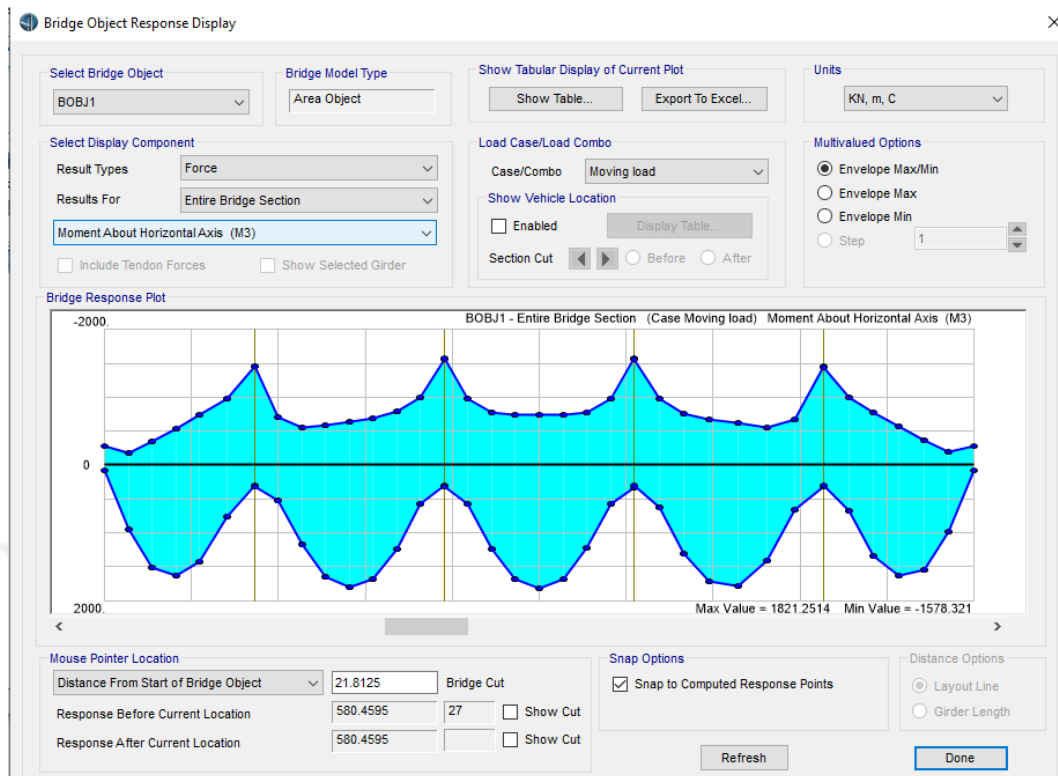


Figure 4.6: Moving Load Moment.

4.2.2 Result Type Stress

The results of the stress analysis performed in the CSI bridge software under the moving load scenario are shown in Figure 4.2.2 for the whole 60-meter bridge section span. Calculated values for the longitudinal stress at the top and bottom center were 2439.81 KN/M^2 and -2132.521 KN/M^2 , respectively.

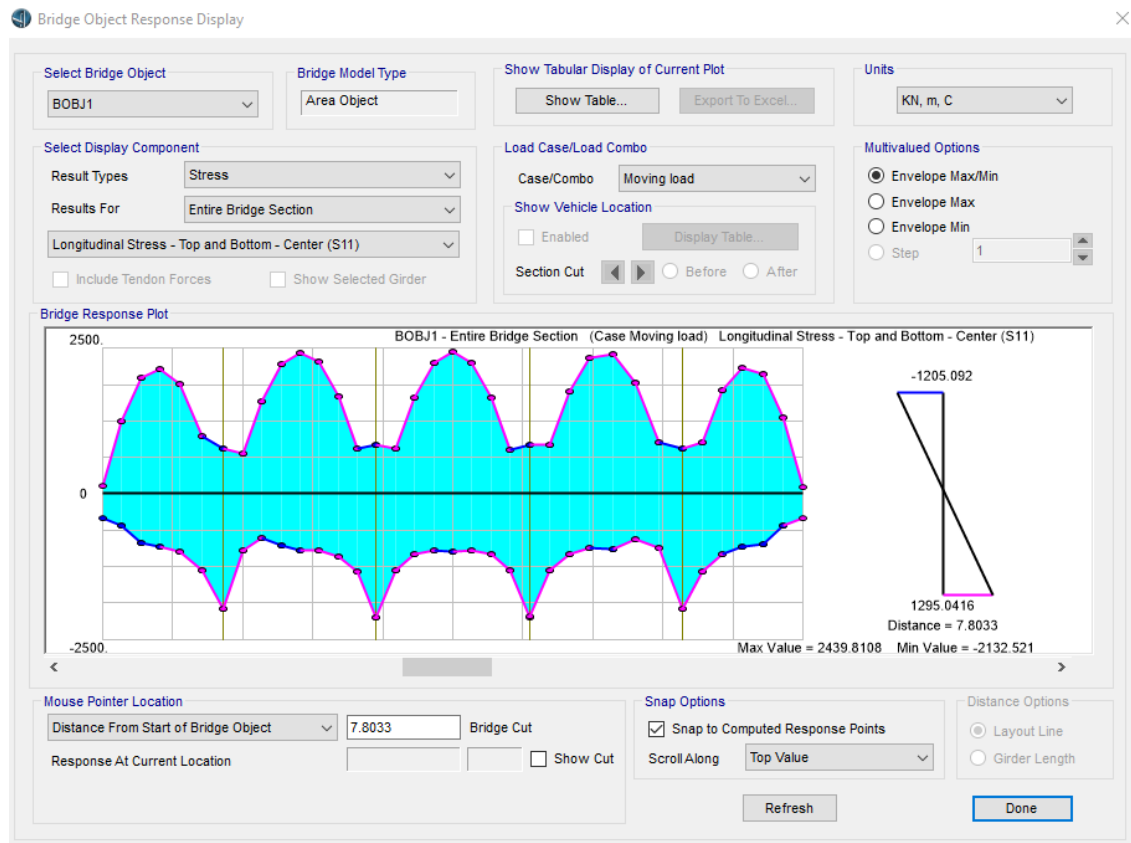


Figure 4.7: Moving Load Stress.

4.2.3 Result Type Displacement

4.2.3.1 Vertical displacement

The reinforcement concrete bridge analysis results are shown in the case of a moving load, result type displacement. Figure 4.2.3 shows the vertical displacement at the entire bridge section, which has a length of 60 meters. The maximum vertical displacement was measured at $9.486\text{E-}04$ meters and the minimum at $-2.442\text{E-}03$ meters, respectively.

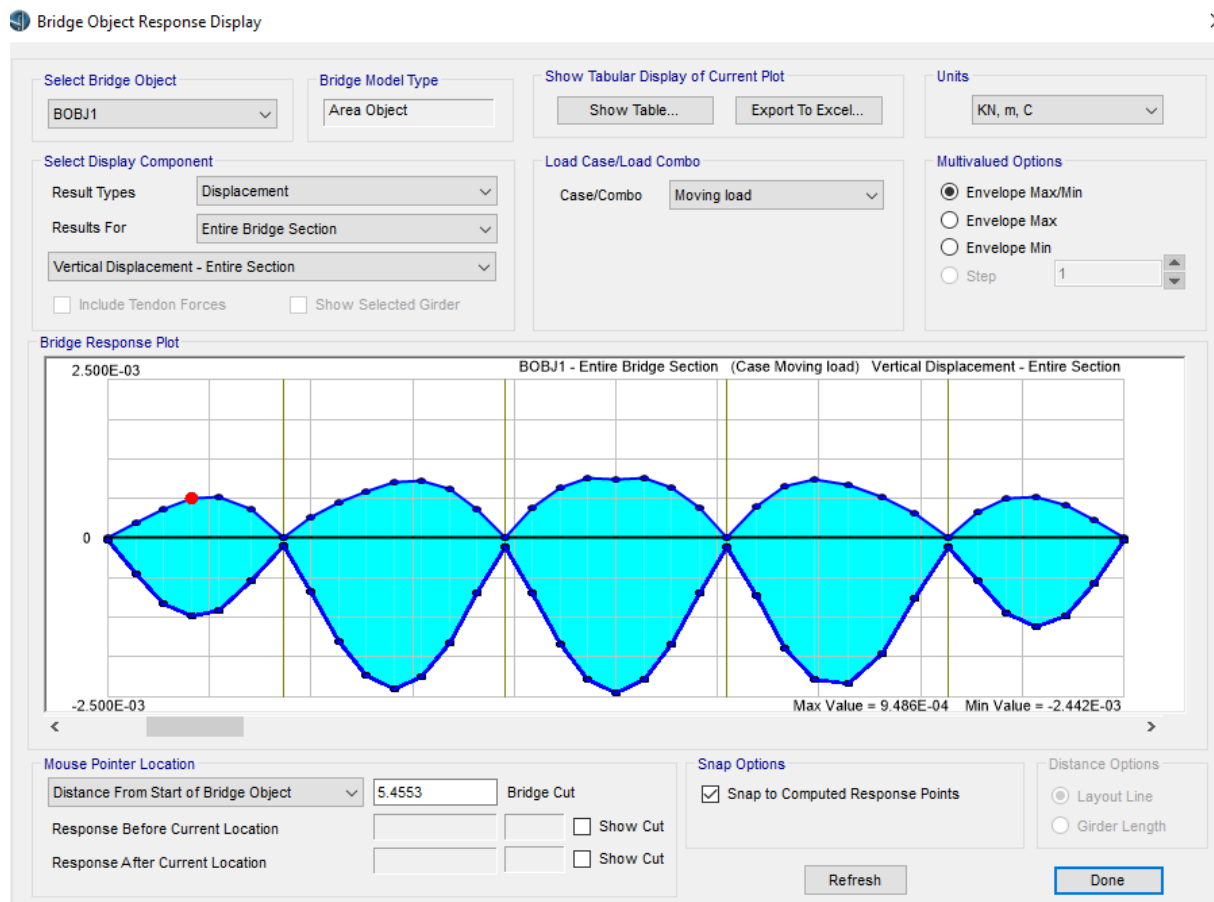


Figure 4.8: Moving Load Vertical Displacement.

4.2.3.2 Longitudinal displacement

The results of the reinforcement concrete bridge analysis for a moving load and result type displacement are shown. Figure 4.2.3 shows the longitudinal displacement at the entire bridge section, which has a length of 60 meters. The largest longitudinal displacement was 4.155×10^{-4} meters, and the smallest was -2.251×10^{-4} meters.

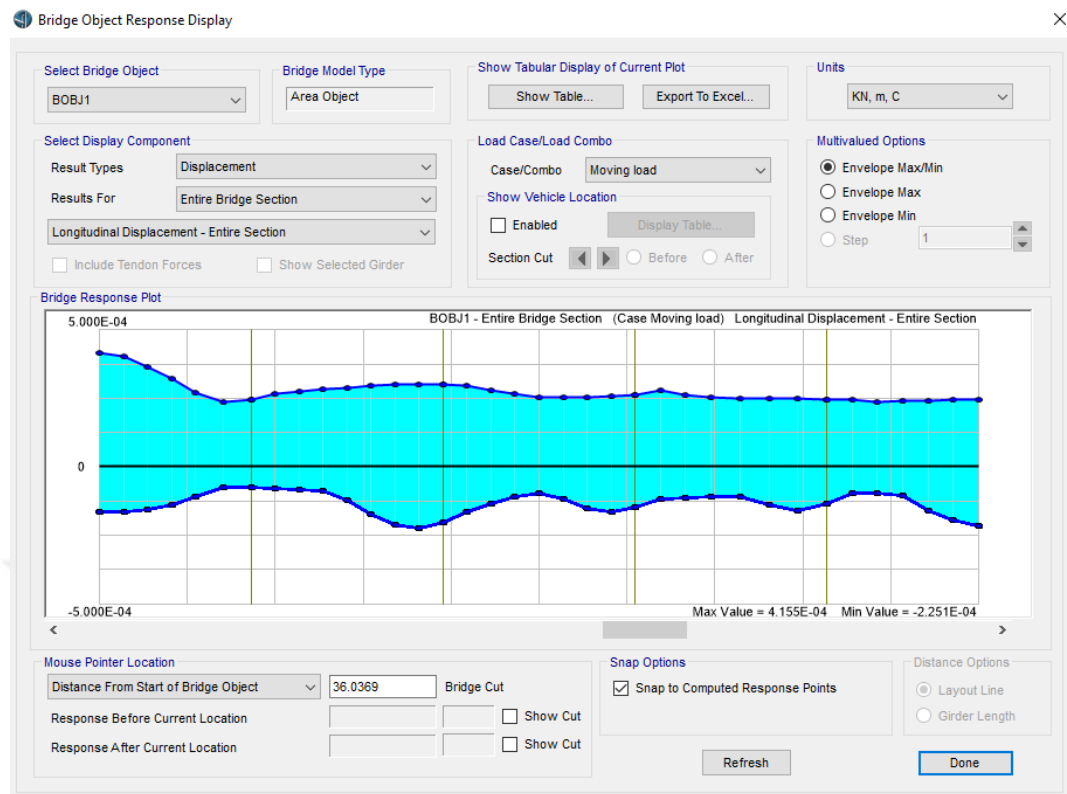


Figure 4.9: Moving Load Longitudinal Displacement.

4.3 TRUCKS LOAD CASE

4.3.1 Result Type Force

In the case of a trucks load, result type force, the reinforcement concrete bridge analysis results are shown in. Figure 4.3.1 shows the moment about the horizontal axis for the entire bridge section, which has a length of 60 meters. The maximum moment about the horizontal axis was calculated as 1049.76 KN-M, and the minimum moment about the horizontal axis was -964.93 KN-M.

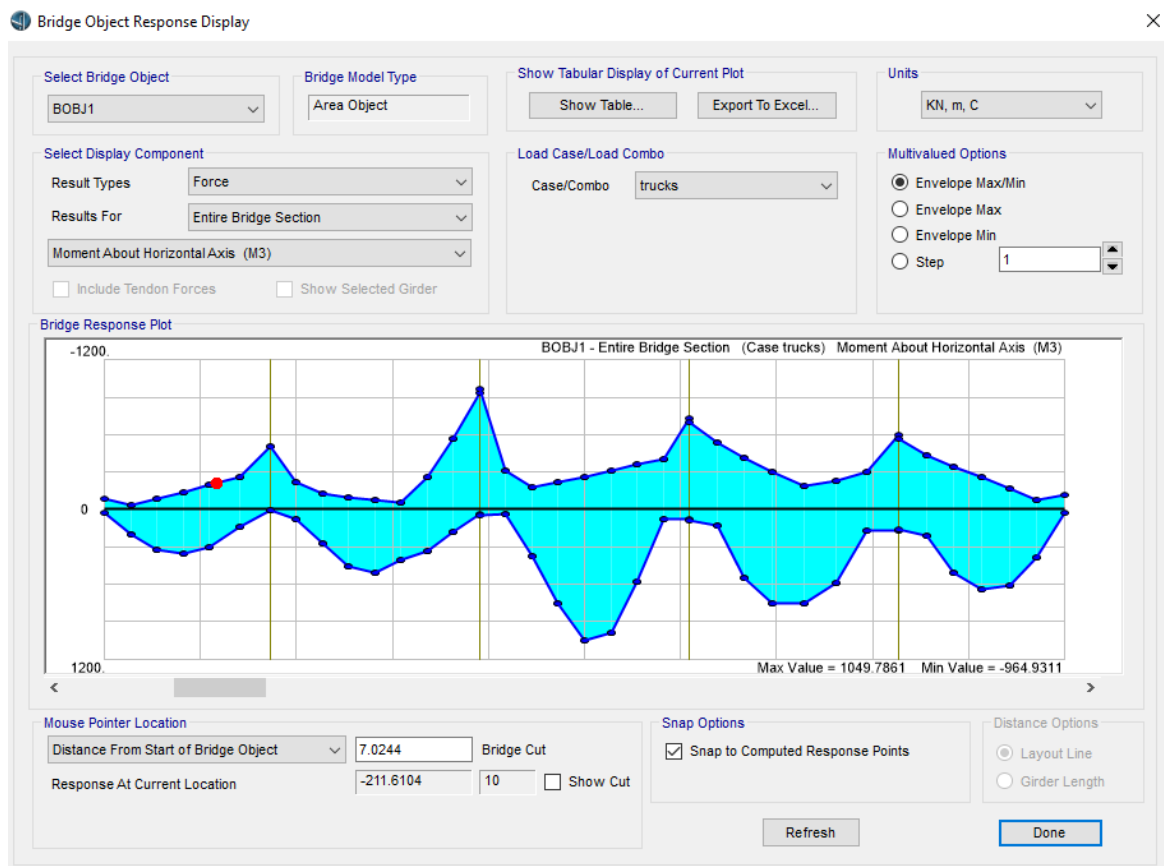


Figure 4.10: Trucks Load Moment.

4.3.2 Result Type Stress

The results of the stress analysis performed in the CSI bridge software under the trucks load scenario are shown in Figure 4.3.2 for the whole 60-meter bridge section span. Calculated values for the longitudinal stress at the top and bottom center were $1411.22 \text{ KN}/\text{M}^2$ and $-1305.66 \text{ KN}/\text{M}^2$, respectively.

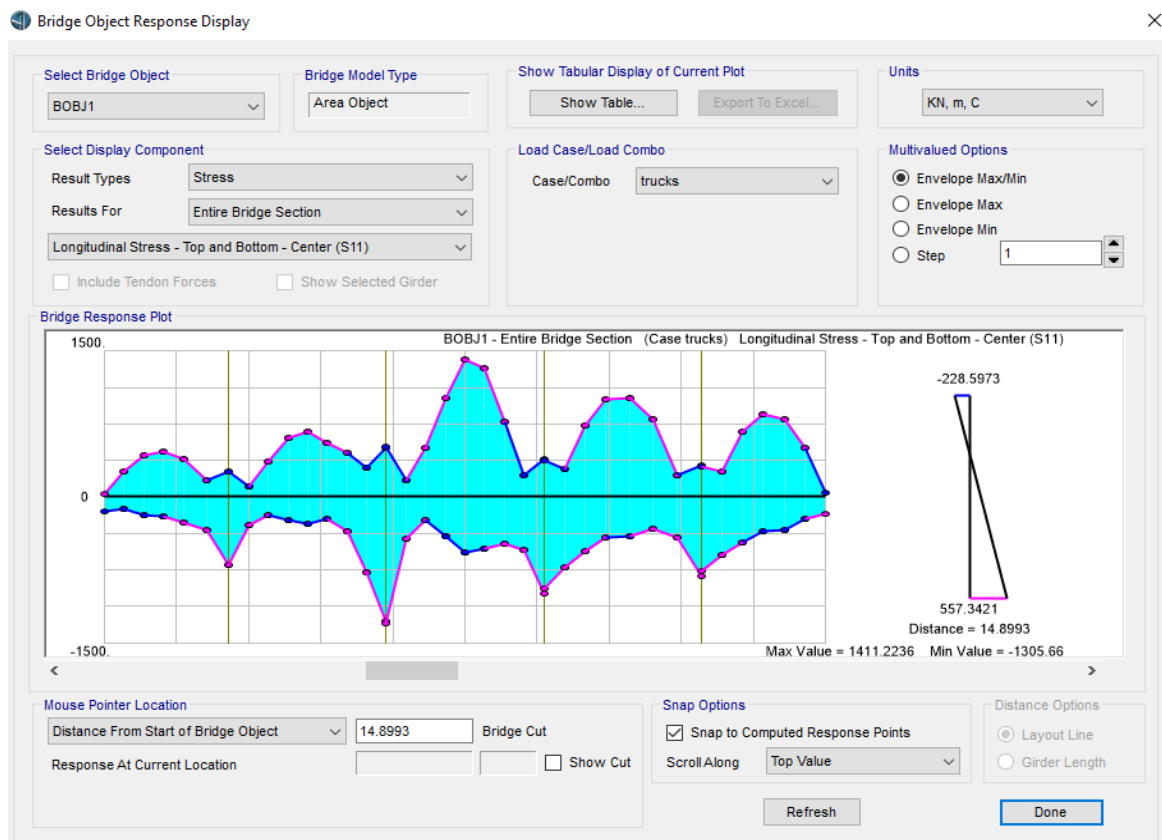


Figure 4.11: Trucks Load Stress.

4.3.3 Result Type Displacement

4.3.3.1 Vertical displacement

The reinforcement concrete bridge analysis results are shown in the case of a truck load, result type displacement. Figure 4.3.3.1 shows the vertical displacement at the entire bridge section, which has a length of 60 meters. The maximum vertical displacement was measured at $3.389\text{E-}04$ meters and the minimum at $-9.585\text{E-}04$ meters, respectively.

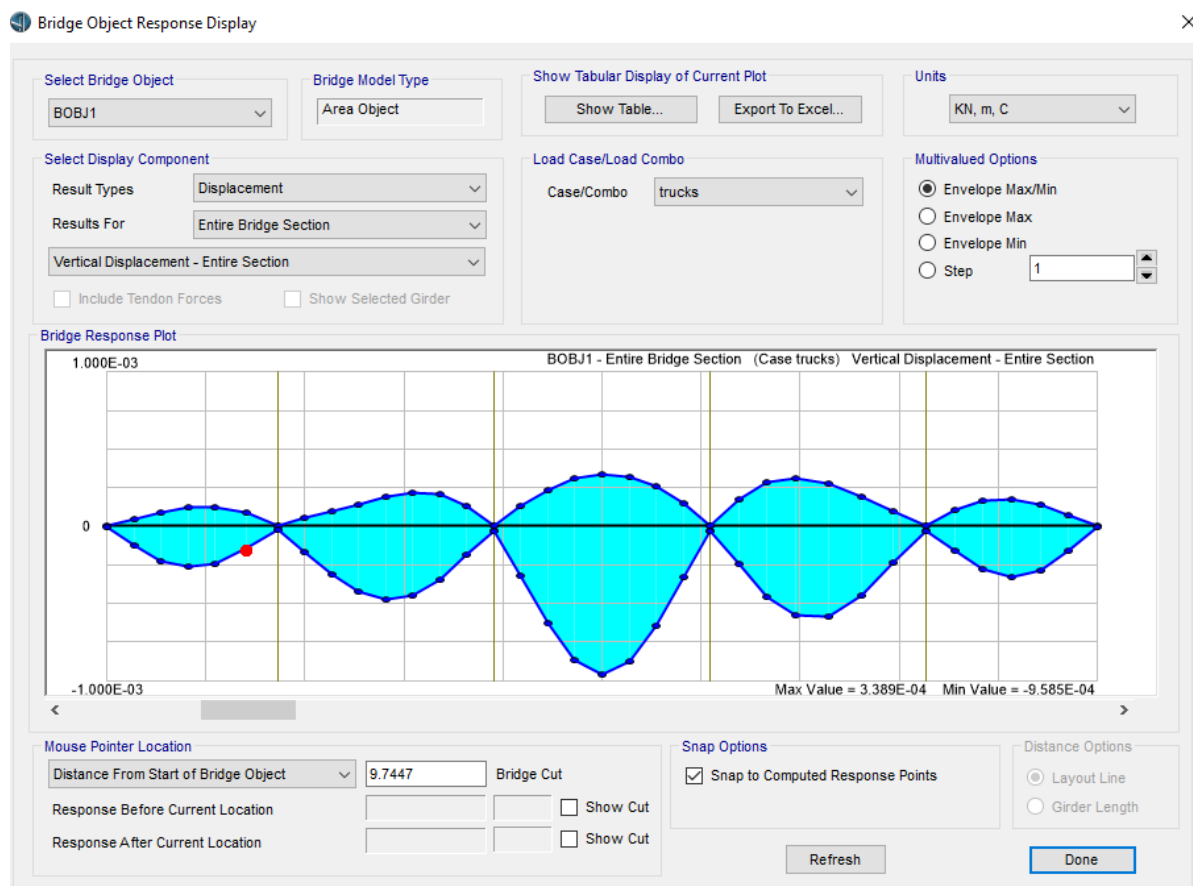


Figure 4.12: Truck Load Vertical Displacement.

4.3.3.2 Longitudinal displacement

The results of the reinforcement concrete bridge analysis for a truck load and result type displacement are shown. Figure 4.3.3.2 shows the longitudinal displacement at the entire bridge section, which has a length of 60 meters. The largest longitudinal displacement was $8.134\text{E-}05$ meters, and the smallest was $-6.110\text{E-}05$ meters.

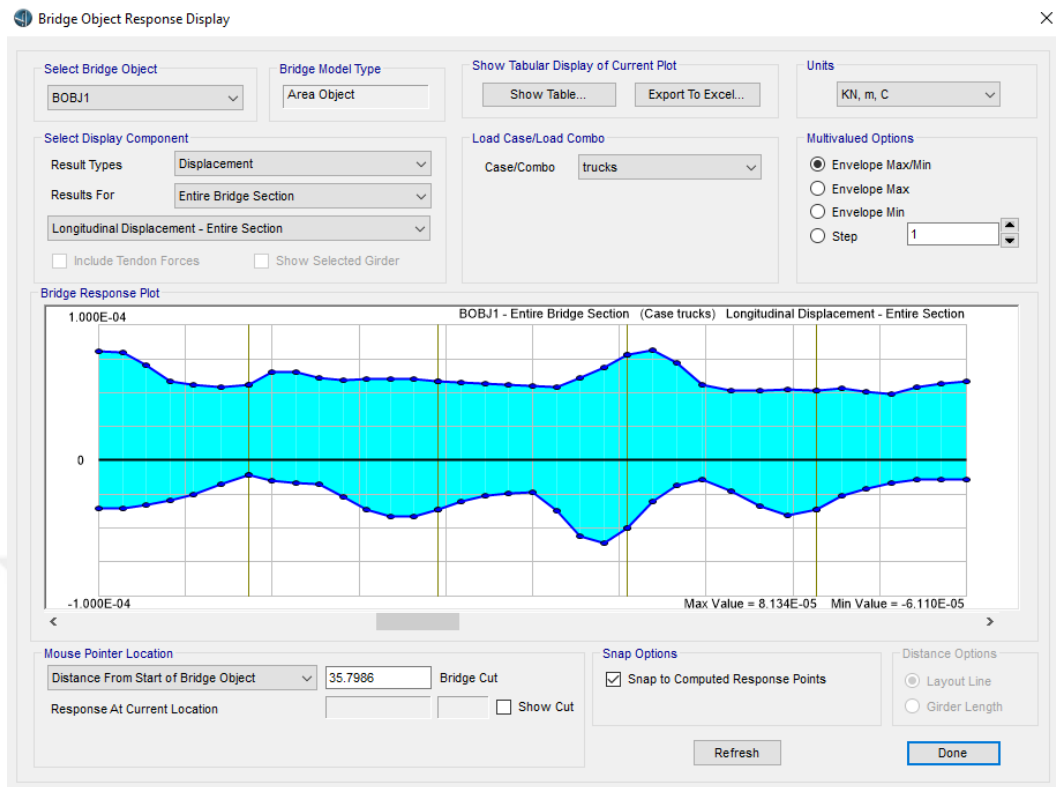


Figure 4.13: Truck Load Longitudinal Displacement.

4.4 COMB 1 LOAD RESULTS

The comb1 consist of asphalt load with factor of 1.5, borders load with factor of 1.25, moving load factor of 1.75 and pedestrian load factor of 1.75 and railing load factor of 1.25 the comb1 loads were also display

Load Combination Data

Load Combination Name: (User-Generated) COMB1

Notes: Modify/Show Notes...

Load Combination Type: Linear Add

Options: Convert to User Load Combo Create Nonlinear Load Case from Load Combo

Define Combination of Load Case Results

Load Case Name	Load Case Type	Scale Factor
Asphalts	Linear Static	1.5
Asphalts	Linear Static	1.5
Borders	Linear Static	1.25
Moving load	Moving Load	1.75
Railings	Linear Static	1.25
Pedestrains	Linear Static	1.75

Add Modify Delete

OK Cancel

Figure 4.14: Load Comb1.

4.4.1 Result Type Force

In the case of a combo1 load, result type force, the reinforcement concrete bridge analysis results are shown in. Figure 4.4.1 shows the moment about the horizontal axis for the entire bridge section, which has a length of 60 meters. The maximum moment about the horizontal axis was calculated as 3520.03 KN-M, and the minimum moment about the horizontal axis was -3439.06 KN-M.

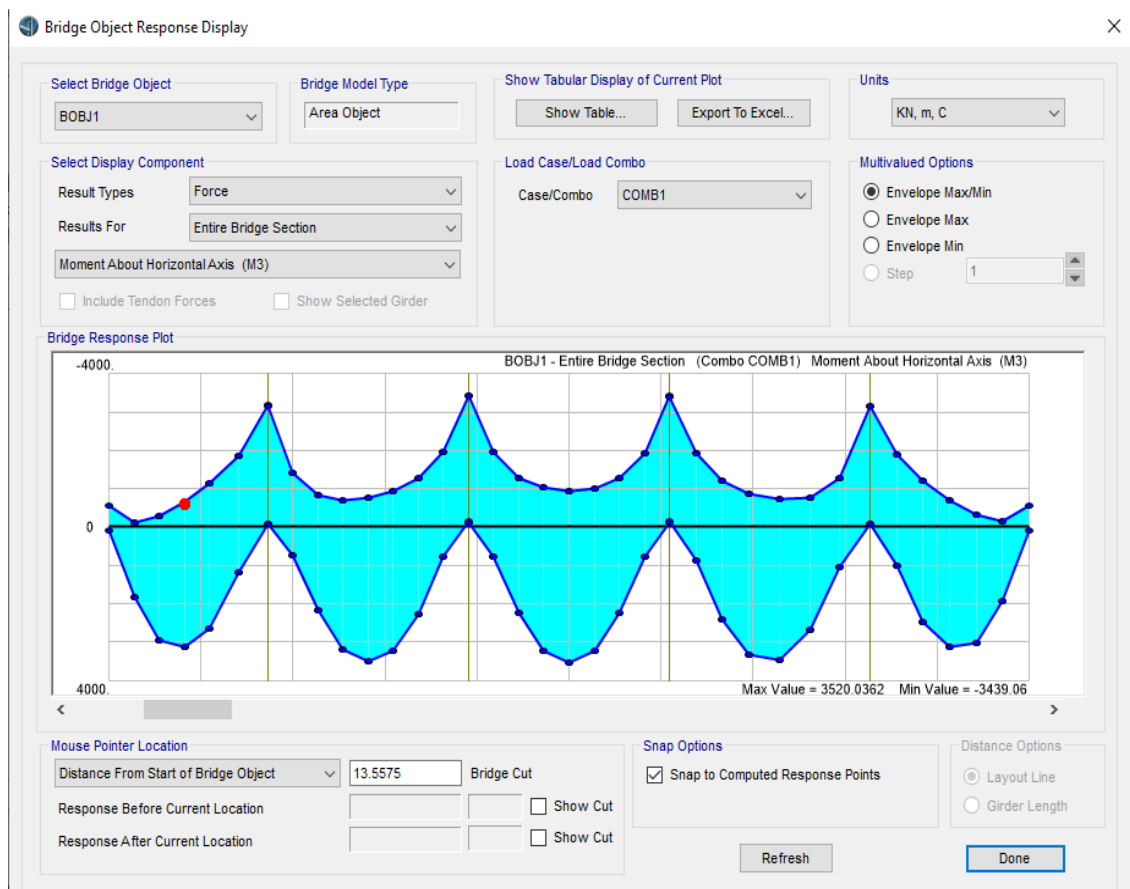


Figure 4.15: Load Comb1 Moment.

4.4.2 Result Type Stress

The results of the stress analysis performed in the CSI bridge software under the combo1 load scenario are shown in Figure 4.4.2 for the whole 60-meter bridge section span. Calculated values for the longitudinal stress at the top and bottom center were 4726.90 KN/M^2 and --4655.42 KN/M^2 , respectively.

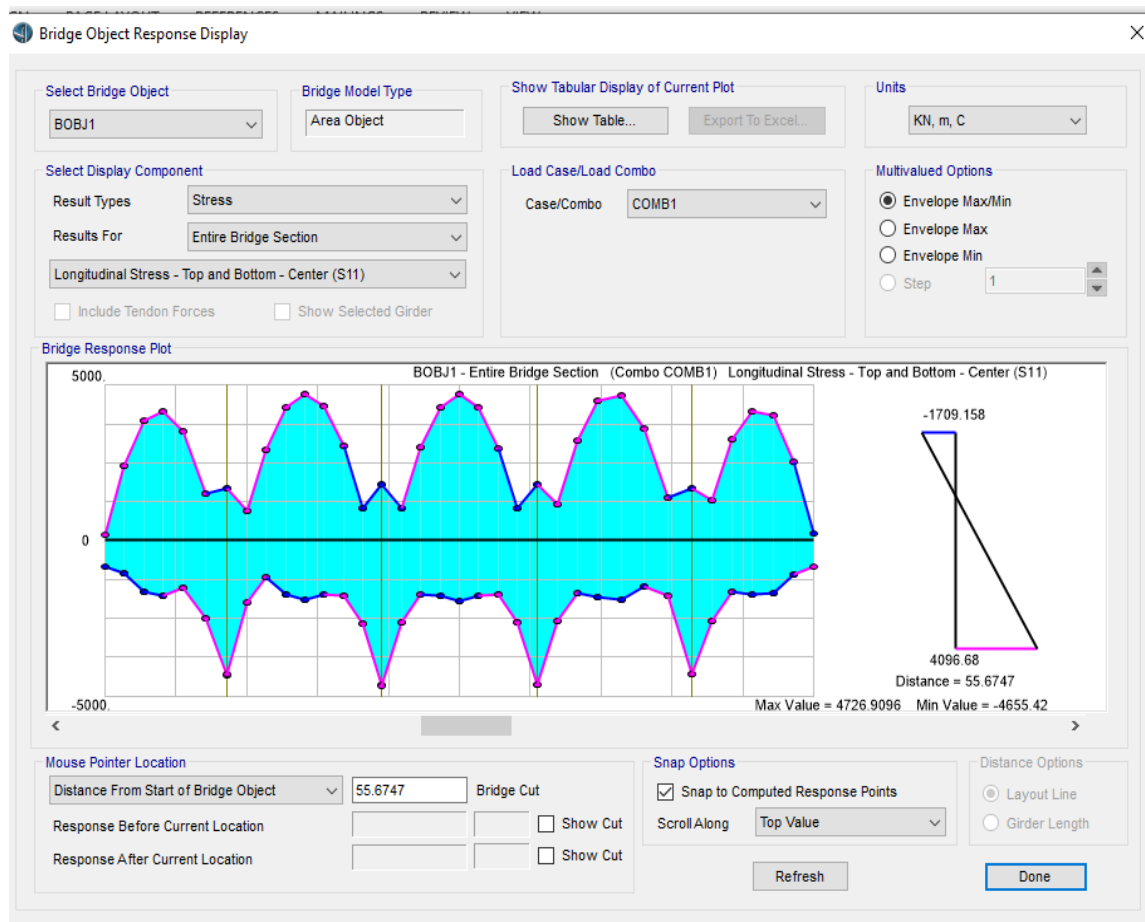


Figure 4.16:Load Comb1 Stress.

4.4.3 Result Type Displacement

4.4.3.1 Vertical displacement

The reinforcement concrete bridge analysis results are shown in the case of a combo1 load, result type displacement. Figure 4.4.3.1 shows the vertical displacement at the entire bridge section, which has a length of 60 meters. The maximum vertical displacement was measured at $1.197\text{E-}03$ meters and the minimum at $-4.783\text{E-}03$ meters, respectively.

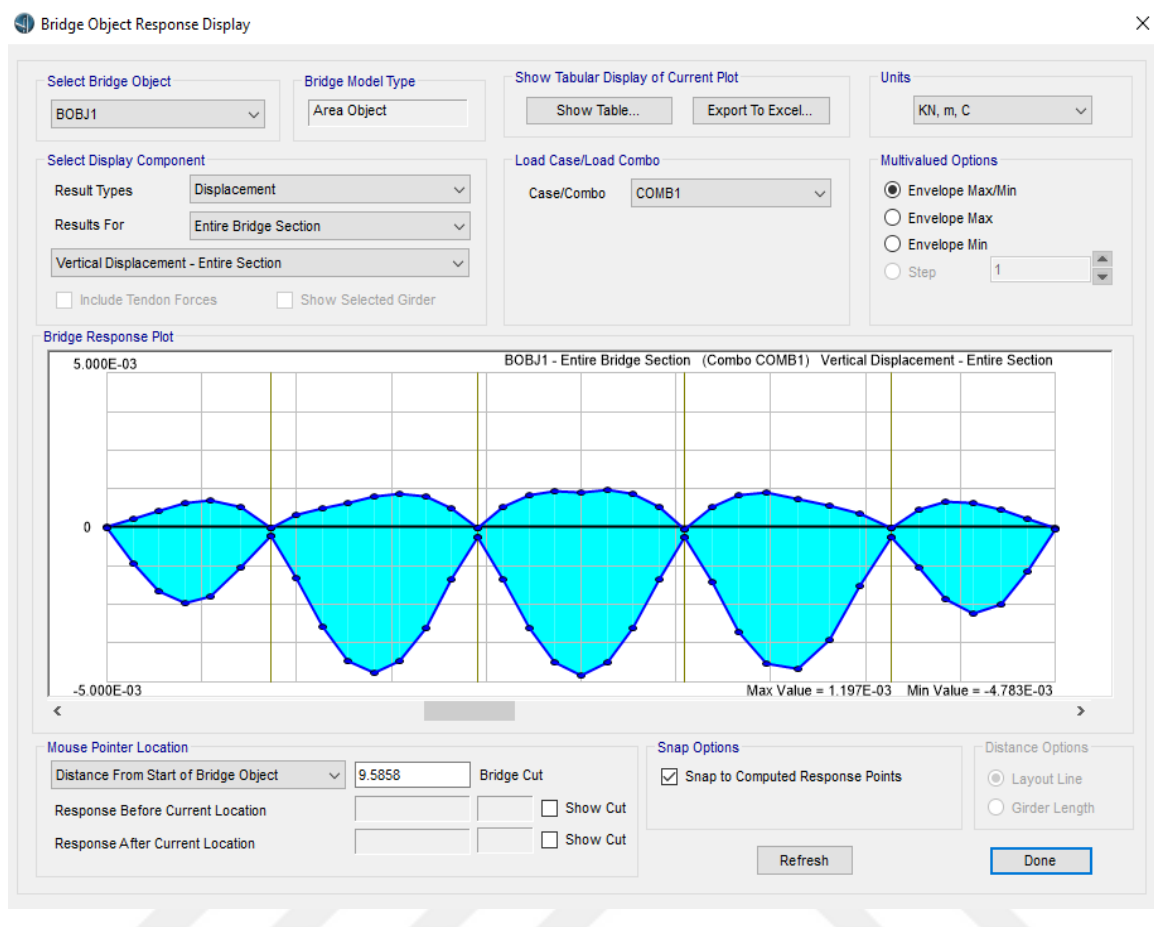


Figure 4.17: Load Comb1 Vertical Displacement.

4.4.3.2 Longitudinal displacement

The results of the reinforcement concrete bridge analysis for a combo1 load and result type displacement are shown. Figure 4.4.3.2 shows the longitudinal displacement at the entire bridge section, which has a length of 60 meters. The largest longitudinal displacement was 8.196E-04meters, and the smallest was -3.799E-04 meters.

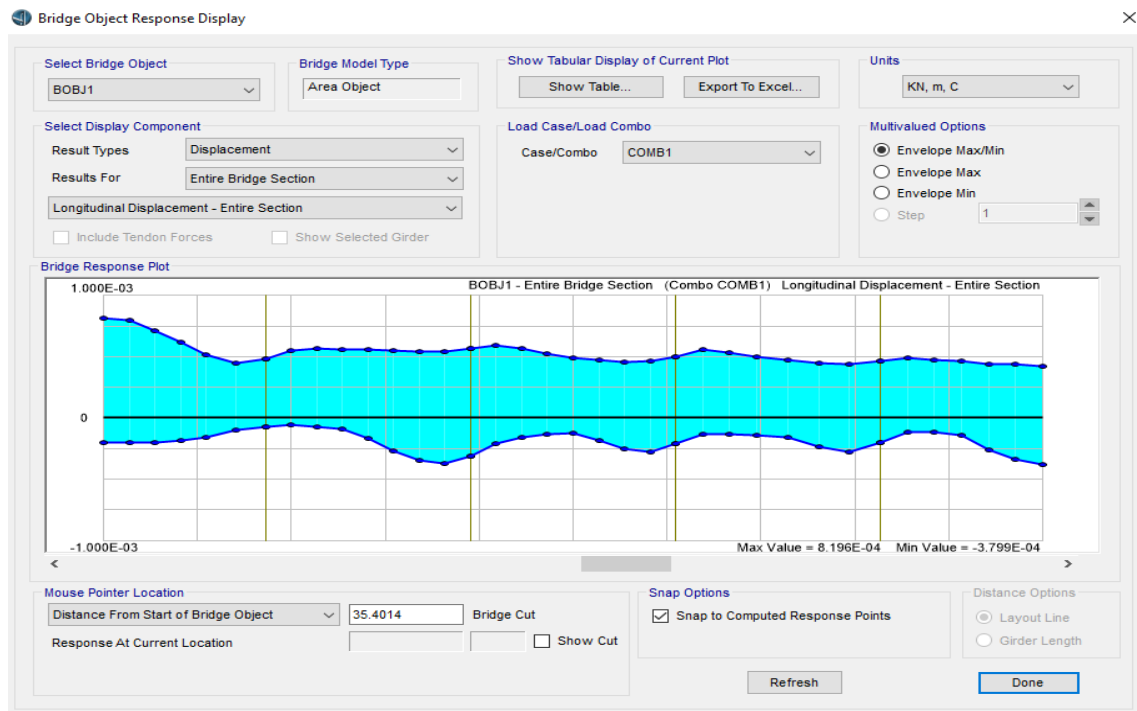


Figure 4.18: Load Comb1 Longitudinal Displacement.

4.5 MODAL ANALYSIS RESULTS

The results of the modal analysis are used to evaluate the natural frequencies and periods of any structure that may be effected by seismic activity. In order to figure out seismic response of proposed bridge first six modes which are represented as distorted forms, as well as their associated frequencies, are shown below.

4.5.1 Mode-1 Results

Figure 4.5.1 shows the deformed shape proposed bridge in mode 1 at time $T = 0.2666$ seconds and frequency $F = 3.75096$ Hz

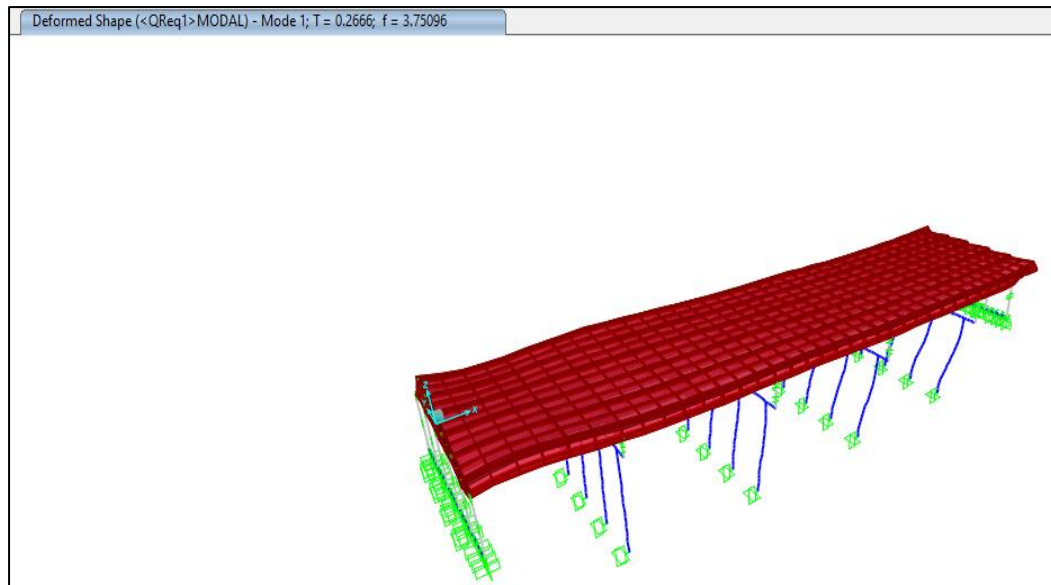


Figure 4.19: Mode-1.

4.5.2 Mode-2

Figure 4.5.2 shows the deformed shape proposed bridge in mode 2 at time $T = 0.15572$ seconds and frequency $F = 6.4216$ Hz

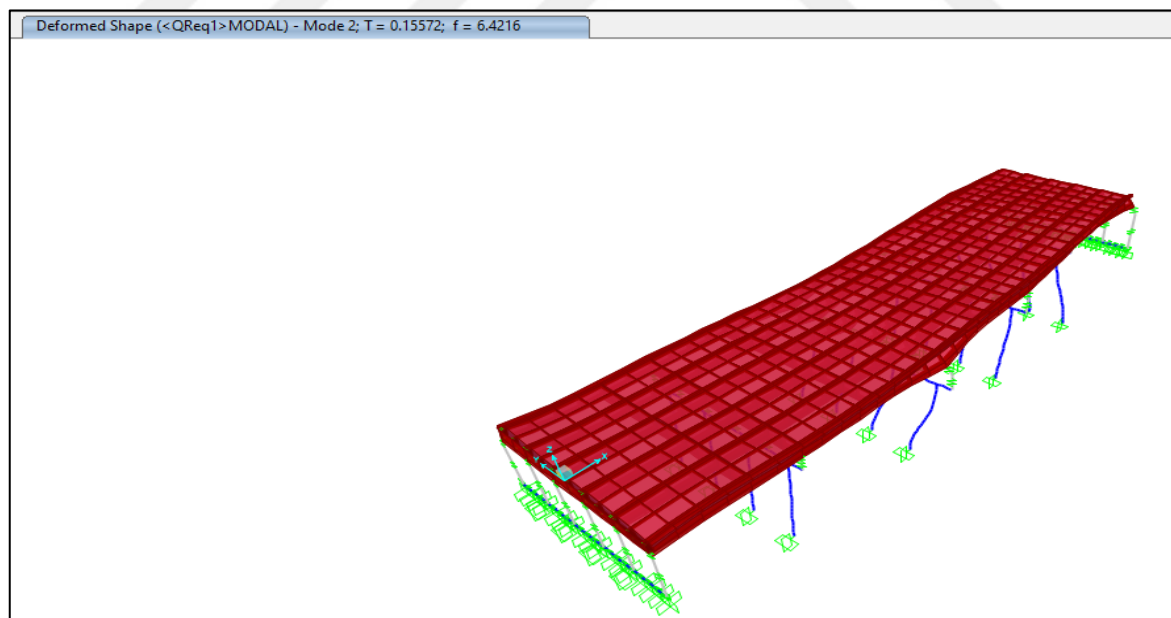


Figure 4.20: Mode-2.

4.5.3 Mode-3

Figure 4.5.3 shows the deformed shape proposed bridge in mode 3 at time $T = 0.15567$ seconds and frequency $F = 6.42367$ Hz.

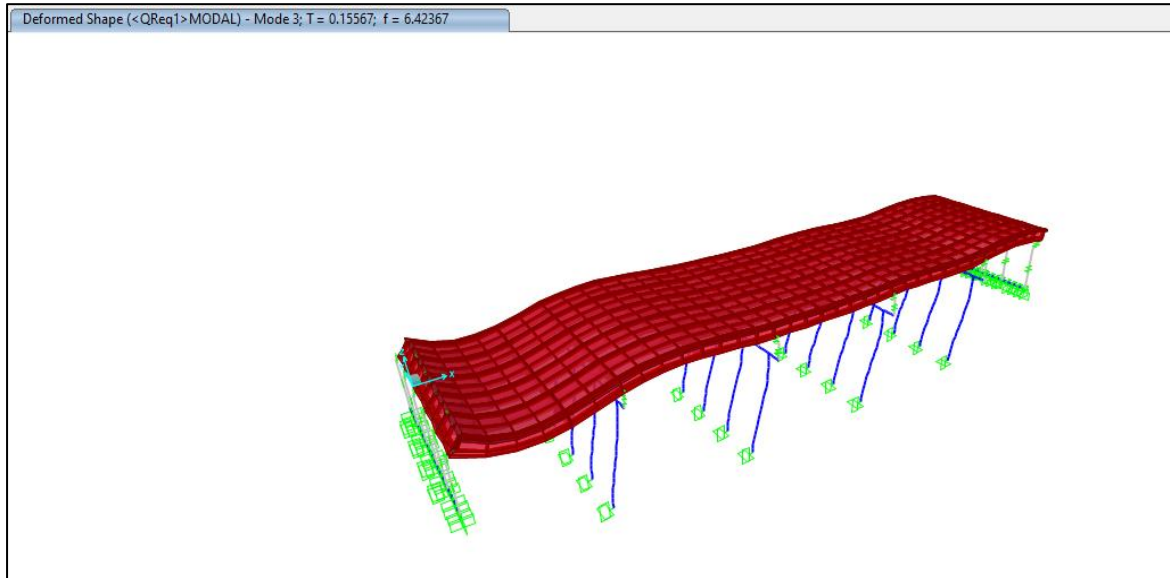


Figure 4.21: Mode-3.

4.5.4 Mode-4

Figure 4.5.4 shows the deformed shape proposed bridge in mode 4, $T = 0.13542$ seconds and $F = 7.3843$ Hz

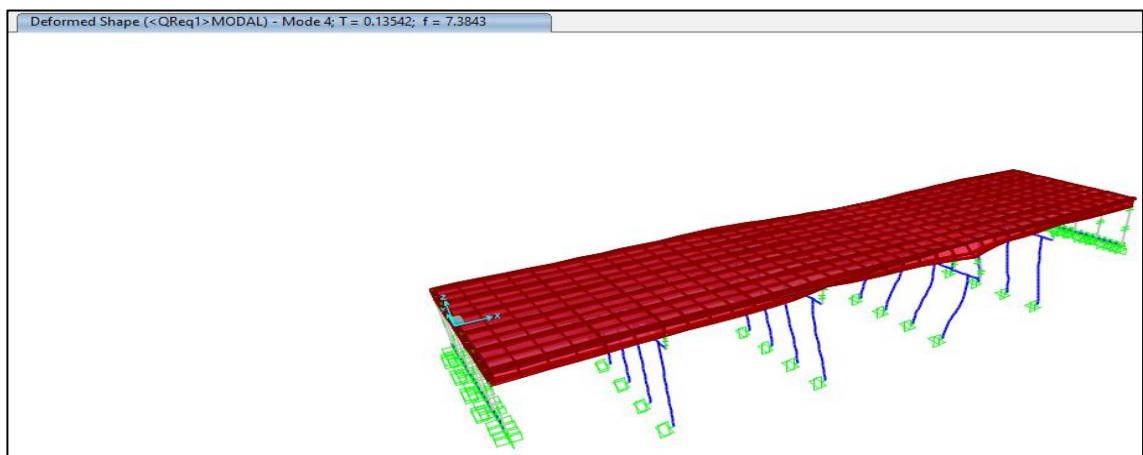


Figure 4.22: Mode-4.

4.5.5 Mode-5

Figure 4.5.5 shows the deformed shape proposed bridge in mode 5, $T = 0.12669$ seconds and $F = 7.89323$ Hz

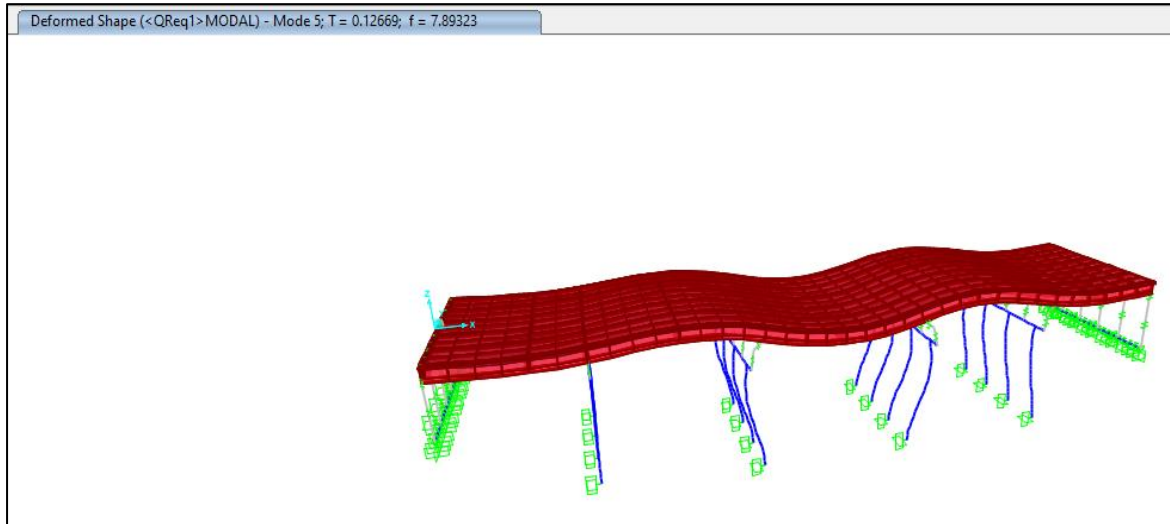


Figure 4. 23: Mode-5.

4.5.6 Mode-6

Figure 4.5.6 shows the deformed shape proposed bridge in mode 6, $T = 0.12199$ seconds and $F = 8.19727$ Hz

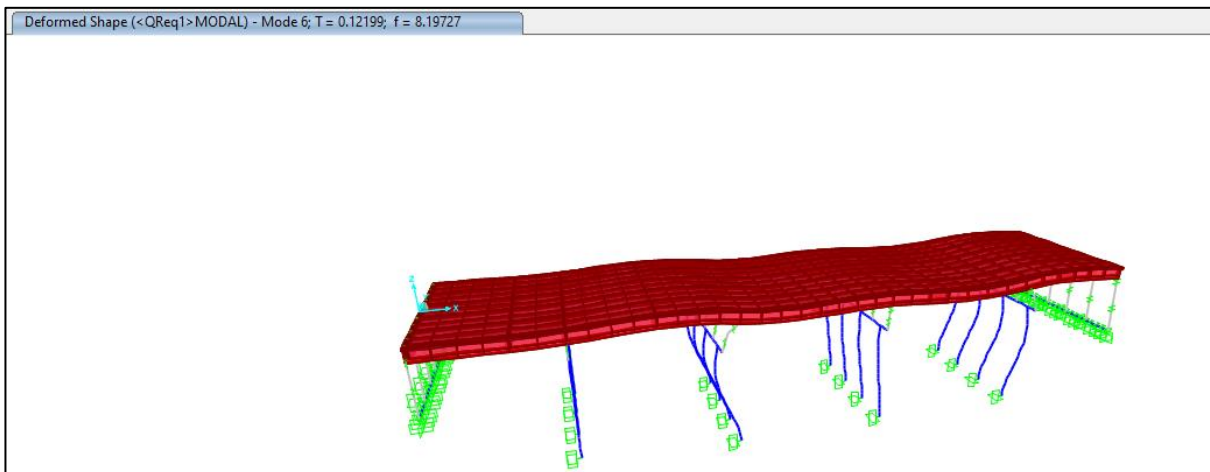


Figure 4.24: Mode-6.

4.6 PUSH OVER ANALYSIS RESULT

The CSi Bridge software determines the displacement capacity by performing a pushover analysis. In the case of proposed bridge the seismic design category is D. The displacement capacity is determined by taking the point on the curve that is situated immediately after the peak value. Following are pushover curves, which were obtain after seismic analysis indicate the displacement capabilities of the bents in both directions

4.6.1 Pushover Curve 1

Pushover curve 1 shows the result of Load case-1 PO_TR1 for bent 1 in transverse direction the shows the displacement of 38.66mm

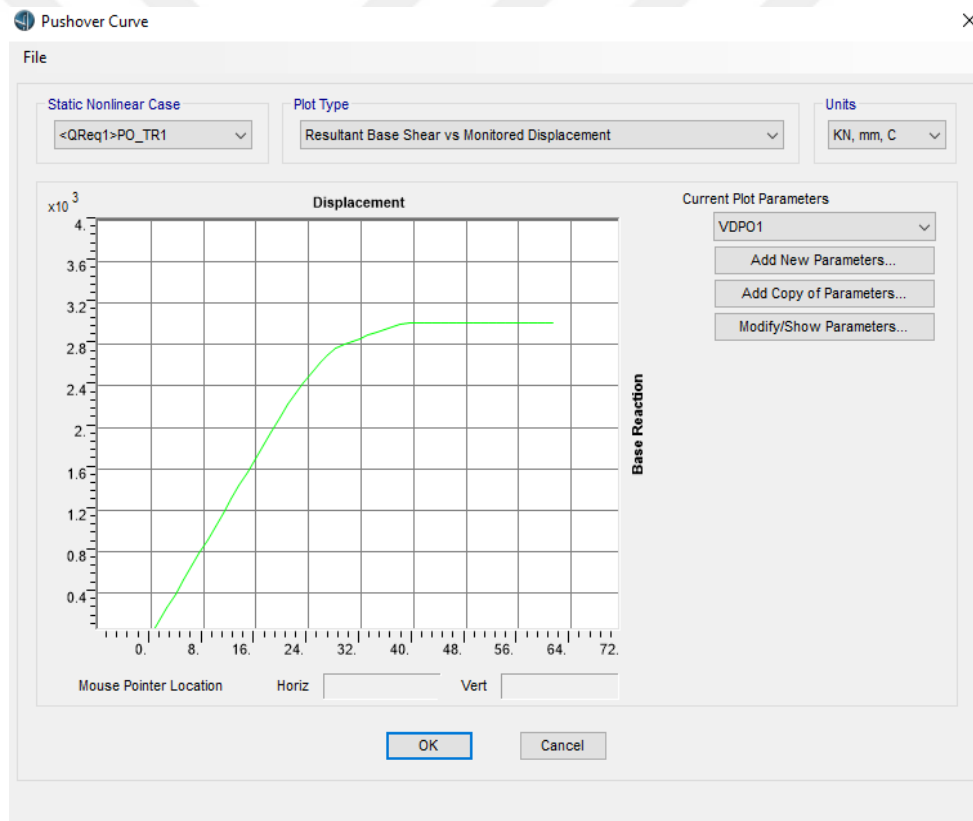


Figure 4.25: Pushover Curve 1.

4.6.2 Pushover Curve 2

Pushover curve 2 shows Load case 2 PO_LR1 for bent 1 in longitudinal direction which shows the displacement of 43.25mm.

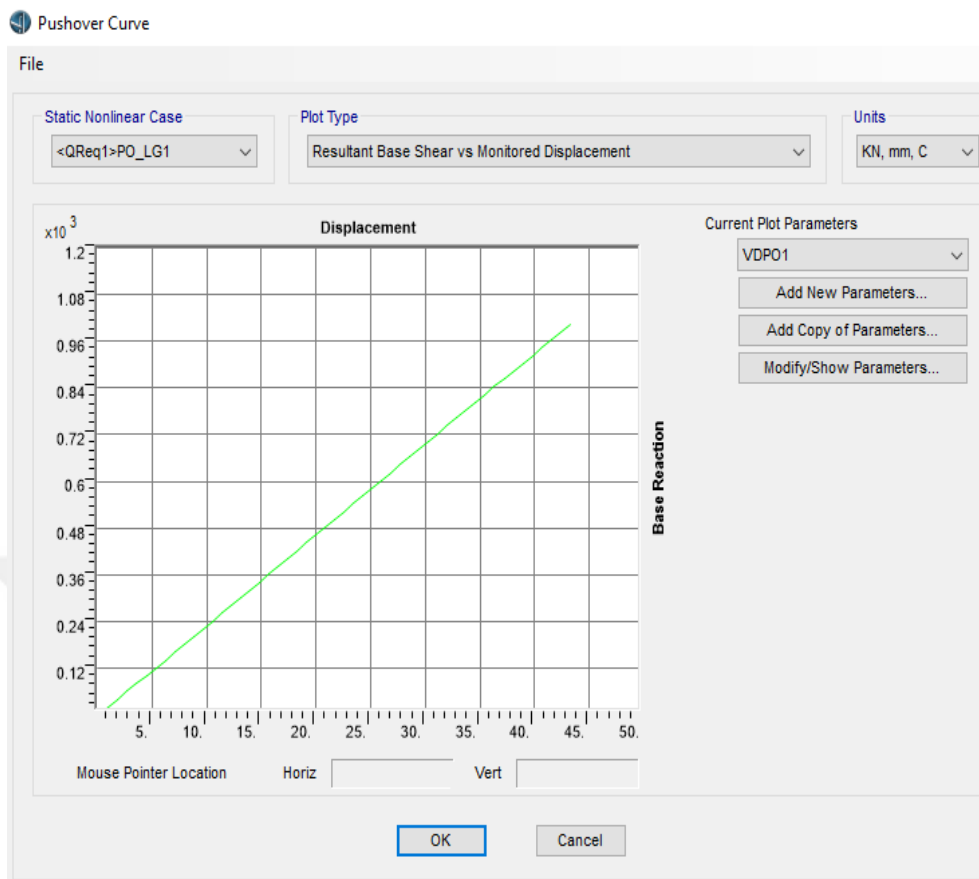


Figure 4.26: Pushover Curve 2.

4.6.3 Pushover Curve 3

The pushover curve 3 shows the result of load case 3 PO_TR2 for bent 2 in transverse direction which shows the displacement of 39.08mm



Figure 4.27: Pushover Curve 3.

4.6.4 Pushover Curve 4

Pushover curve 4 show the displacement of 57.04ment m the loadcase4 PO_LR for bent 2 in longitudinal direction

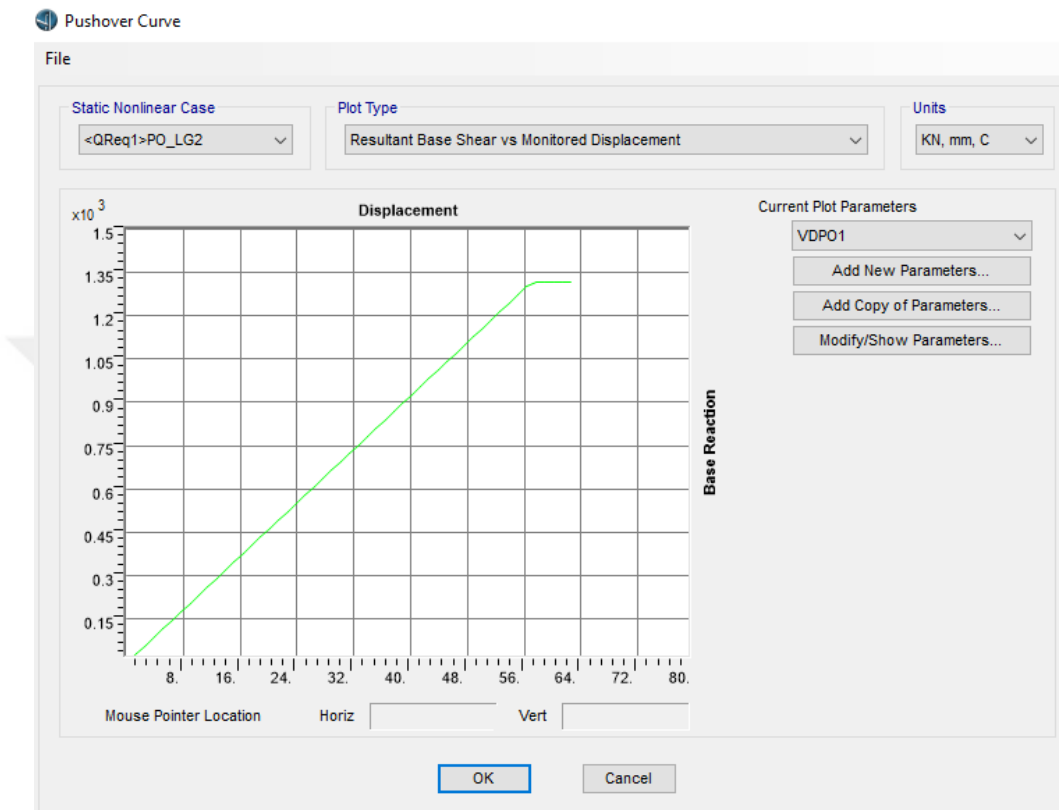


Figure 4.28: Pushover Curve 4.

4.6.5 Pushover Curve 5

Load case 5 PO_TR for bent 3 in transverse direction shows the displacement of 41.78mm in pushover curve 5

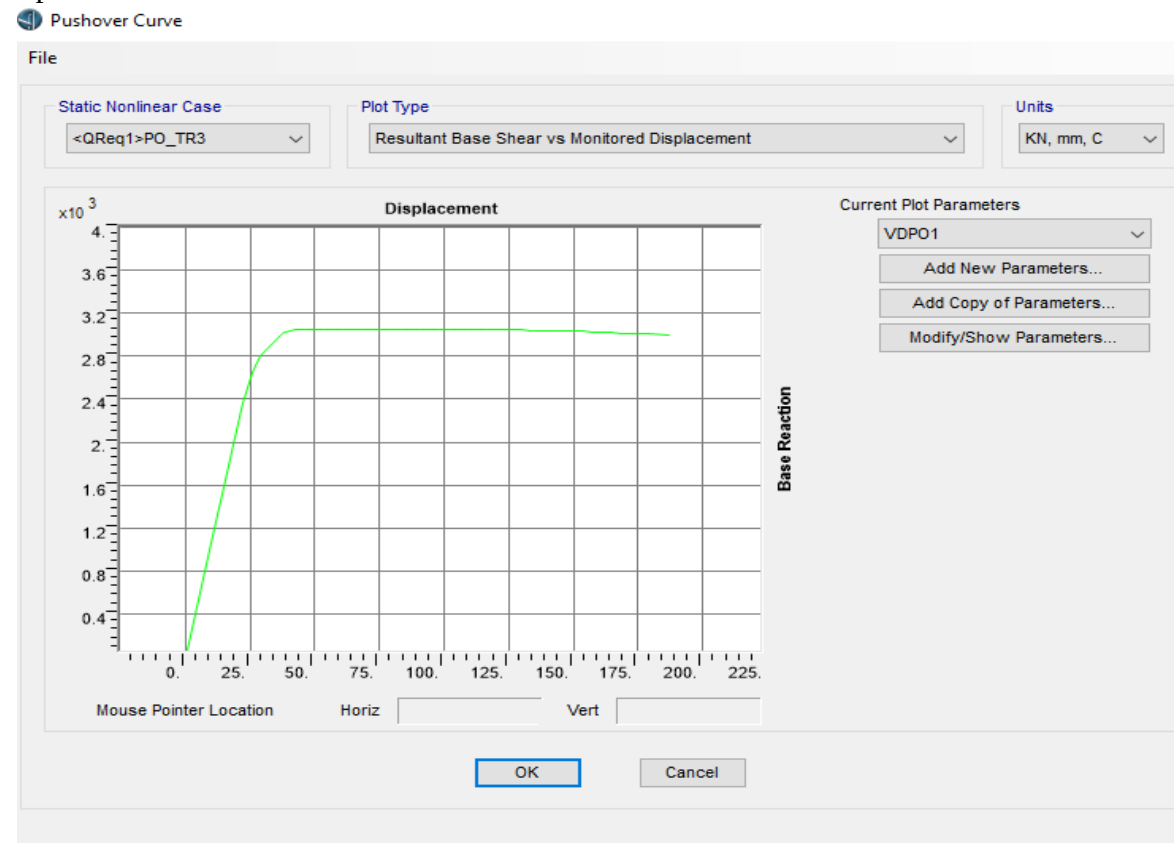


Figure 4.29: Pushover Curve 5.

4.6.6 Pushover Curve 6

In this Pushover curve 6 the load case 6 PO_LG3 for bent 3 in longitudinal direction show the displacement of 57.07mm

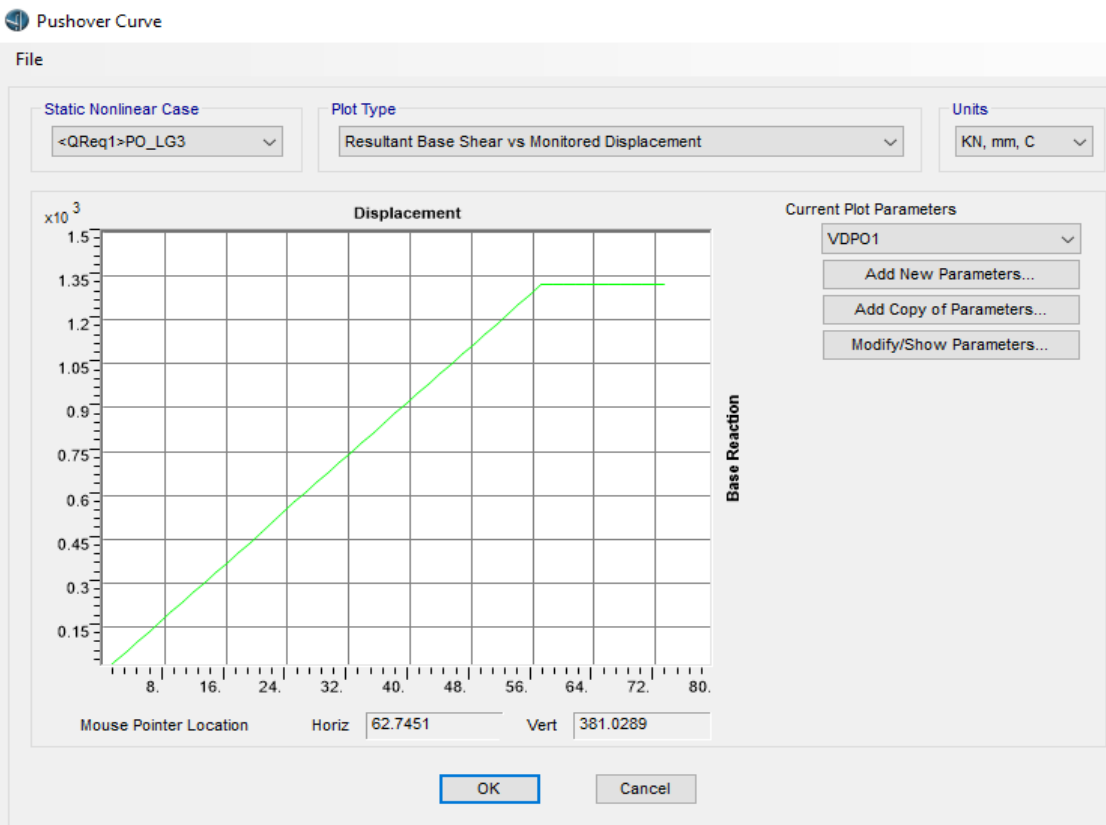


Figure 4.30: Pushover Curve 6.

4.6.7 Pushover Curve 7

The pushover curve 7 shows the result of load case 7 PO_TR4 for bent 4 in transverse direction the shows the displacement of 42.27mm

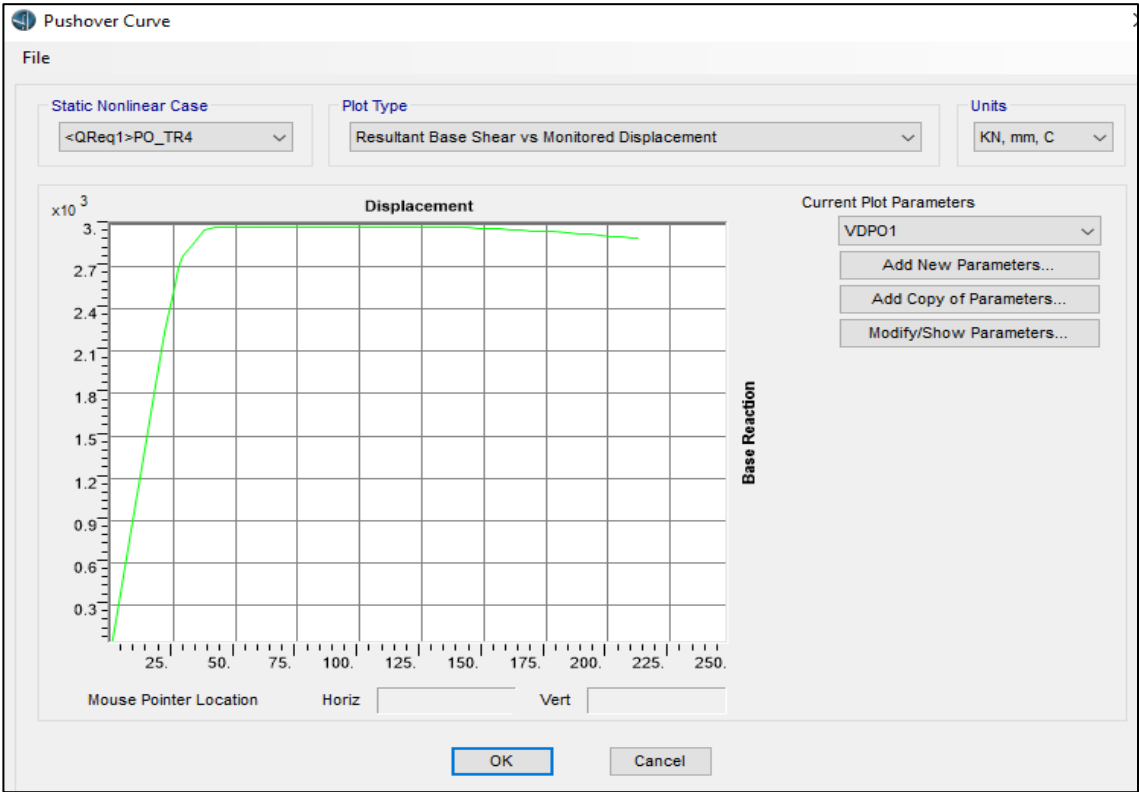


Figure 4.31: Pushover Curve 7.

4.6.8 Pushover Curve 8

In this Pushover curve 8 the load case 8 PO_LG4 for bent 4 in longitudinal direction show the displacement of 57.25mm

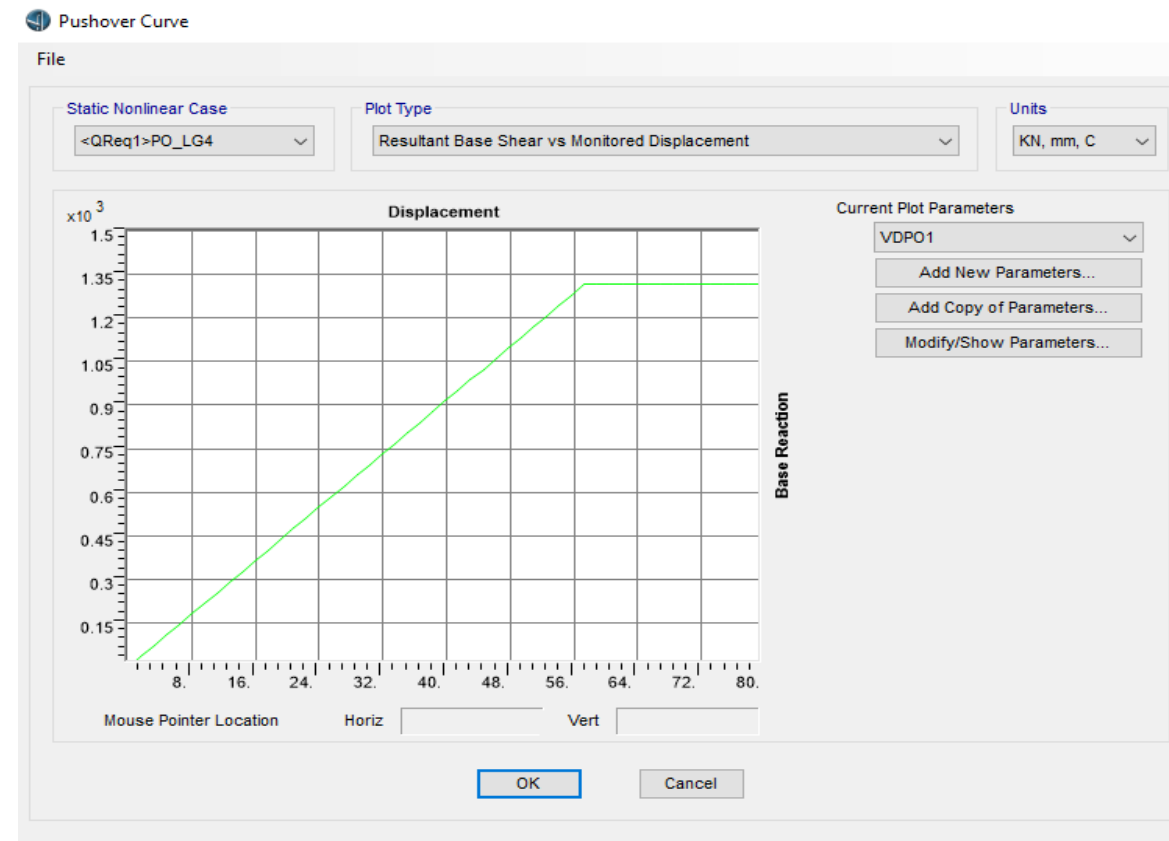


Figure 4.32: Pushover Curve 8.

4.7 BENT DEMAND/ CAPACITY’S RATIO

The following table provides information on the demand-to-capacity ratios for each kind of bend in each possible direction. When the value is less than 1, it implies that there is sufficient capacity for the ground motion hazard being utilized in the seismic design request for the particular bend and direction being considered. A state of overstress was indicated by values that were more than 1.0.

Table 4.1: Bridge Seismic - Bent D-C.

DesReq Name	BridgeObj	SDCategory	Span Name	Station	Direction	GenDispl	Demand	Capacity	DCR ratio
				M			m	m	
Seismic	BOBJ1	D	Span 2	10.35	TRANS	<Seismic> GD_TR1	0.015312	0.061247	0.25
Seismic	BOBJ1	D	Span 2	10.35	LONG	<Seismic> GD_LG1	0.010766	0.043236	0.249
Seismic	BOBJ1	D	Span 3	23.45	TRANS	<Seismic> GD_TR2	0.033855	0.135422	0.25
Seismic	BOBJ1	D	Span 3	23.45	LONG	<Seismic> GD_LG2	0.015605	0.062612	0.2492
Seismic	BOBJ1	D	Span 4	36.55	TRANS	<Seismic> GD_TR3	0.046695	0.170627	0.2737
Seismic	BOBJ1	D	Span 4	36.55	LONG	<Seismic> GD_LG3	0.018266	0.073206	0.2495
Seismic	BOBJ1	D	Span 5	49.65	TRANS	<Seismic> GD_TR4	0.052987	0.171949	0.3082
Seismic	BOBJ1	D	Span 5	49.65	LONG	<Seismic> GD_LG4	0.019832	0.079517	0.2494

4.8 BENT COLUMN FORCE DEMAND

The bent column seismic demand forces are reported in the following table.

Table 4.2: Bridge Seismic - Bent Column Force Demand.

DesReqName	BridgeObj	SpanName	Station	Column	Location	P	V2	V3	T	M2
			M			KN	KN	KN	KN-m	KN-m
Seismic	BOBJ1	Span 2	10.35	1	Top	1315.5	350.914	157.713	1483.0424	813.9464
Seismic	BOBJ1	Span 2	10.35	1	Bottom	1322.259	515.756	511.402	1483.0424	1756.4304
Seismic	BOBJ1	Span 2	10.35	2	Top	1205.022	342.758	104.304	1487.5269	726.7865
Seismic	BOBJ1	Span 2	10.35	2	Bottom	1209.645	505.19	475.951	1487.5269	1602.9567
Seismic	BOBJ1	Span 2	10.35	3	Top	1205.023	342.758	104.305	1487.5252	726.7911
Seismic	BOBJ1	Span 2	10.35	3	Bottom	1209.646	505.189	475.947	1487.5252	1602.9507
Seismic	BOBJ1	Span 2	10.35	4	Top	1315.504	350.916	157.718	1483.0394	813.944
Seismic	BOBJ1	Span 2	10.35	4	Bottom	1322.264	515.757	511.404	1483.0394	1756.4359

Table 4.2: Bridge Seismic - Bent Column Force Demand”Table Continued”.

Seismic	BOBJ1	Span 3	23.45	1	Top	1535.641	703.109	172.67 2	861.88 73	1261.7 736
Seismic	BOBJ1	Span 3	23.45	1	Bottom	1538.798	1058.689	660.07 9	861.88 73	2343.9 239
Seismic	BOBJ1	Span 3	23.45	2	Top	298.006	693.409	150.30 9	884.84 86	1204.0 72
Seismic	BOBJ1	Span 3	23.45	2	Bottom	300.23	1051.316	645.26	884.84 86	2269.8 092
Seismic	BOBJ1	Span 3	23.45	3	Top	298.004	693.408	150.30 9	884.85 4	1204.0 744
Seismic	BOBJ1	Span 3	23.45	3	Bottom	300.228	1051.317	645.25 8	884.85 4	2269.8 038
Seismic	BOBJ1	Span 3	23.45	4	Top	1535.645	703.11	172.67 1	861.89 71	1261.7 71
Seismic	BOBJ1	Span 3	23.45	4	Bottom	1538.802	1058.688	660.08 3	861.89 71	2343.9 306
Seismic	BOBJ1	Span 4	36.55	1	Top	1619.119	989.578	197.53 4	724.91 37	1425.6 356
Seismic	BOBJ1	Span 4	36.55	1	Bottom	1622.257	1441.295	766.31	724.91 37	2702.9 318
Seismic	BOBJ1	Span 4	36.55	2	Top	349.988	975.605	174.07 4	752.07 14	1374.5 56
Seismic	BOBJ1	Span 4	36.55	2	Bottom	351.319	1432.755	743.09 1	752.07 14	2620.5 075
Seismic	BOBJ1	Span 4	36.55	3	Top	349.986	975.611	174.07 4	752.07 03	1374.5 555
Seismic	BOBJ1	Span 4	36.55	3	Bottom	351.317	1432.749	743.09 1	752.07 03	2620.5 089
Seismic	BOBJ1	Span 4	36.55	4	Top	1619.131	989.577	197.53 3	724.91 17	1425.6 355
Seismic	BOBJ1	Span 4	36.55	4	Bottom	1622.269	1441.296	766.31	724.91 17	2702.9 322
Seismic	BOBJ1	Span 5	49.65	1	Top	2311.193	1106.39	207.22 9	537.16 37	1526.5 659
Seismic	BOBJ1	Span 5	49.65	1	Bottom	2315.433	1624.064	820.486	537.163 7	2885.02 2
Seismic	BOBJ1	Span 5	49.65	2	Top	424.646	1095.319	185.011	547.487 4	1487.38 45
Seismic	BOBJ1	Span 5	49.65	2	Bottom	429.658	1619.158	802.145	547.487 4	2824.50 92
Seismic	BOBJ1	Span 5	49.65	3	Top	424.644	1095.322	185.016	547.489 9	1487.39 12
Seismic	BOBJ1	Span 5	49.65	3	Bottom	429.656	1619.154	802.14	547.489 9	2824.49 94
Seismic	BOBJ1	Span 5	49.65	4	Top	2311.194	1106.389	207.228	537.168 1	1526.56 44
Seismic	BOBJ1	Span 5	49.65	4	Bottom	2315.433	1624.064	820.487	537.168 1	2885.02 34

4.9 BENT COLUMN IDEALIZED MOMENT CAPACITY'S

The idealized column plastic moments were computed, and the results found in the table that follows. The demand axial load is represented by the axial load letter P. It is possible to calculate the idealized plastic moments by making use of the related axial load value, P.

Table 4.3: Bridge Seismic - Bent Column Idealized Moment Capacity.

DesReqName	BridgeObj	SpanName	Station m	Column	Location	GenHinge	P KN	Mp2 KN-m	Mp3 KN-m
Seismic	BOBJ1	Span 2	10.35	1	Top	4H1	-979.956	2637.9075	2637.9075
Seismic	BOBJ1	Span 2	10.35	1	Top	4H2	-979.956	2637.9075	2637.9075
Seismic	BOBJ1	Span 2	10.35	1	Bottom	4H1	-1459.642	2957.2886	2957.2886
Seismic	BOBJ1	Span 2	10.35	1	Bottom	4H2	-1459.642	2957.2886	2957.2886
Seismic	BOBJ1	Span 2	10.35	2	Top	5H1	-916.801	2597.1048	2597.1048
Seismic	BOBJ1	Span 2	10.35	2	Top	5H2	-916.801	2597.1048	2597.1048
Seismic	BOBJ1	Span 2	10.35	2	Bottom	5H1	-1396.488	2914.2423	2914.2423
Seismic	BOBJ1	Span 2	10.35	2	Bottom	5H2	-1396.488	2914.2423	2914.2423
Seismic	BOBJ1	Span 2	10.35	3	Top	6H1	-916.801	2597.1048	2597.1048
Seismic	BOBJ1	Span 2	10.35	3	Top	6H2	-916.801	2597.1048	2597.1048
Seismic	BOBJ1	Span 2	10.35	3	Bottom	6H1	-1396.488	2914.2423	2914.2423
Seismic	BOBJ1	Span 2	10.35	3	Bottom	6H2	-1396.488	2914.2423	2914.2423
Seismic	BOBJ1	Span 2	10.35	4	Top	7H1	-979.956	2637.9075	2637.9075
Seismic	BOBJ1	Span 2	10.35	4	Top	7H2	-979.956	2637.9075	2637.9075
Seismic	BOBJ1	Span 2	10.35	4	Bottom	7H1	-1459.642	2957.2886	2957.2886
Seismic	BOBJ1	Span 2	10.35	4	Bottom	7H2	-1459.642	2957.2886	2957.2886
Seismic	BOBJ1	Span 3	23.45	1	Top	9H1	-983.818	2640.3869	2640.3869
Seismic	BOBJ1	Span 3	23.45	1	Top	9H2	-983.818	2640.3869	2640.3869
Seismic	BOBJ1	Span 3	23.45	1	Bottom	9H1	-1463.505	2959.9853	2959.9853
Seismic	BOBJ1	Span 3	23.45	1	Bottom	9H2	-1463.505	2959.9853	2959.9853
Seismic	BOBJ1	Span 3	23.45	2	Top	10H1	-873.846	2568.5038	2568.5038
Seismic	BOBJ1	Span 3	23.45	2	Top	10H2	-873.846	2568.5038	2568.5038
Seismic	BOBJ1	Span 3	23.45	2	Bottom	10H1	-1353.533	2885.7042	2885.7042
Seismic	BOBJ1	Span 3	23.45	2	Bottom	10H2	-1353.533	2885.7042	2885.7042
Seismic	BOBJ1	Span 3	23.45	3	Top	11H1	-873.846	2568.5038	2568.5038
Seismic	BOBJ1	Span 3	23.45	3	Top	11H2	-873.846	2568.5038	2568.5038
Seismic	BOBJ1	Span 3	23.45	3	Bottom	11H1	-1353.533	2885.7042	2885.7042
Seismic	BOBJ1	Span 3	23.45	3	Bottom	11H2	-1353.533	2885.7042	2885.7042
Seismic	BOBJ1	Span 3	23.45	4	Top	12H1	-983.818	2640.3869	2640.3869
Seismic	BOBJ1	Span 3	23.45	4	Top	12H2	-983.818	2640.3869	2640.3869
Seismic	BOBJ1	Span 3	23.45	4	Bottom	12H1	-1463.505	2959.9853	2959.9853
Seismic	BOBJ1	Span 3	23.45	4	Bottom	12H2	-1463.505	2959.9853	2959.9853
Seismic	BOBJ1	Span 4	36.55	1	Top	14H1	-987.255	2642.5947	2642.5947
Seismic	BOBJ1	Span 4	36.55	1	Top	14H2	-987.255	2642.5947	2642.5947
Seismic	BOBJ1	Span 4	36.55	1	Bottom	14H1	-1466.941	2962.3936	2962.3936
Seismic	BOBJ1	Span 4	36.55	1	Bottom	14H2	-1466.941	2962.3936	2962.3936
Seismic	BOBJ1	Span 4	36.55	2	Top	15H1	-879.126	2572.0243	2572.0243
Seismic	BOBJ1	Span 4	36.55	2	Top	15H2	-879.126	2572.0243	2572.0243
Seismic	BOBJ1	Span 4	36.55	2	Bottom	15H1	-1358.813	2889.241	2889.241
Seismic	BOBJ1	Span 4	36.55	2	Bottom	15H2	-1358.813	2889.241	2889.241
Seismic	BOBJ1	Span 4	36.55	3	Top	16H1	-879.126	2572.0243	2572.0243
Seismic	BOBJ1	Span 4	36.55	3	Top	16H2	-879.126	2572.0243	2572.0243
Seismic	BOBJ1	Span 4	36.55	3	Bottom	16H1	-1358.813	2889.241	2889.241
Seismic	BOBJ1	Span 4	36.55	3	Bottom	16H2	-1358.813	2889.241	2889.241
Seismic	BOBJ1	Span 4	36.55	4	Top	17H1	-987.255	2642.5947	2642.5947
Seismic	BOBJ1	Span 4	36.55	4	Top	17H2	-987.255	2642.5947	2642.5947
Seismic	BOBJ1	Span 4	36.55	4	Bottom	17H1	-1466.941	2962.3936	2962.3936
Seismic	BOBJ1	Span 4	36.55	4	Bottom	17H2	-1466.941	2962.3936	2962.3936
Seismic	BOBJ1	Span 5	49.65	1	Top	19H1	-960.441	2625.4161	2625.4161
Seismic	BOBJ1	Span 5	49.65	1	Top	19H2	-960.441	2625.4161	2625.4161
Seismic	BOBJ1	Span 5	49.65	1	Bottom	19H1	-1440.128	2943.5152	2943.5152
Seismic	BOBJ1	Span 5	49.65	1	Bottom	19H2	-1440.128	2943.5152	2943.5152
Seismic	BOBJ1	Span 5	49.65	2	Top	20H1	-856.476	2556.9261	2556.9261
Seismic	BOBJ1	Span 5	49.65	2	Top	20H2	-856.476	2556.9261	2556.9261
Seismic	BOBJ1	Span 5	49.65	2	Bottom	20H1	-1336.163	2873.6085	2873.6085
Seismic	BOBJ1	Span 5	49.65	2	Bottom	20H2	-1336.163	2873.6085	2873.6085
Seismic	BOBJ1	Span 5	49.65	3	Top	21H1	-856.476	2556.9261	2556.9261
Seismic	BOBJ1	Span 5	49.65	3	Top	21H2	-856.476	2556.9261	2556.9261
Seismic	BOBJ1	Span 5	49.65	3	Bottom	21H1	-1336.163	2873.6085	2873.6085
Seismic	BOBJ1	Span 5	49.65	3	Bottom	21H2	-1336.163	2873.6085	2873.6085
Seismic	BOBJ1	Span 5	49.65	4	Top	22H1	-960.441	2625.4161	2625.4161
Seismic	BOBJ1	Span 5	49.65	4	Top	22H2	-960.441	2625.4161	2625.4161
Seismic	BOBJ1	Span 5	49.65	4	Bottom	22H1	-1440.128	2943.5152	2943.5152
Seismic	BOBJ1	Span 5	49.65	4	Bottom	22H2	-1440.128	2943.5152	2943.5152

4.10 BENT COLUMN SECTION CRACK PROPERTIES

The following table provides a report on the cracked section property modifiers that are applied to each of the bent columns.

Table 4.4: Bridge Seismic Bent Column Cracked Properties.

DesReqName	BridgeObj	SpanName	Station	Column	I22Mod	I33Mod
			m			
Seismic	BOBJ1	Span 2	10.35	1	0.111818	0.111818
Seismic	BOBJ1	Span 2	10.35	2	0.109783	0.109783
Seismic	BOBJ1	Span 2	10.35	3	0.109783	0.109783
Seismic	BOBJ1	Span 2	10.35	4	0.111818	0.111818
Seismic	BOBJ1	Span 3	23.45	1	0.111193	0.111193
Seismic	BOBJ1	Span 3	23.45	2	0.108403	0.108403
Seismic	BOBJ1	Span 3	23.45	3	0.108403	0.108403
Seismic	BOBJ1	Span 3	23.45	4	0.111193	0.111193
Seismic	BOBJ1	Span 4	36.55	1	0.11204	0.11204
Seismic	BOBJ1	Span 4	36.55	2	0.108573	0.108573
Seismic	BOBJ1	Span 4	36.55	3	0.108573	0.108573
Seismic	BOBJ1	Span 4	36.55	4	0.11204	0.11204
Seismic	BOBJ1	Span 5	49.65	1	0.111194	0.111194
Seismic	BOBJ1	Span 5	49.65	2	0.107844	0.107844
Seismic	BOBJ1	Span 5	49.65	3	0.107844	0.107844
Seismic	BOBJ1	Span 5	49.65	4	0.111194	0.111194

4.11 SUPPORT BEARING DEMAND - DEFORMATIONS

The following table reports the deformations that occurred in all of the bearings located at the abutments and bents that are resistant to seismic loads.

Table 4.5: Bridge Seismic Design - Support Bearing Demand – Deformations.

DesReq Name	BridgeObj	SpanName	Station	Type	Bearing	U1	U2	U3	R1	R2
			M			m	m	m	Degrees	Degrees
Seismic	BOBJ1	Start Abutment	0.	Abutment	1	1.388E-08	9.843E-09	6.106E-08	0.00343	0.014673
Seismic	BOBJ1	Start Abutment	0.	Abutment	2	5.464E-09	1.615E-08	5.126E-08	0.002442	0.011363
Seismic	BOBJ1	Start Abutment	0.	Abutment	3	3.736E-09	1.687E-08	4.528E-08	0.002481	0.010136
Seismic	BOBJ1	Start Abutment	0.	Abutment	4	3.736E-09	1.687E-08	4.528E-08	0.002481	0.010136
Seismic	BOBJ1	Start Abutment	0.	Abutment	5	5.464E-09	1.615E-08	5.126E-08	0.002442	0.011363
Seismic	BOBJ1	Start Abutment	0.	Abutment	6	1.388E-08	9.843E-09	6.106E-08	0.00343	0.014673
Seismic	BOBJ1	Span 2	10.35	Bent	1	9.238E-09	1.503E-09	5.154E-09	0.000083	0.003188
Seismic	BOBJ1	Span 2	10.35	Bent	2	9.563E-09	3.825E-09	4.673E-09	0.000083	0.003273
Seismic	BOBJ1	Span 2	10.35	Bent	3	8.448E-09	4.260E-09	5.248E-09	0.00008	0.003121
Seismic	BOBJ1	Span 2	10.35	Bent	4	8.448E-09	4.260E-09	5.248E-09	0.00008	0.003121
Seismic	BOBJ1	Span 2	10.35	Bent	5	9.563E-09	3.825E-09	4.673E-09	0.000083	0.003273
Seismic	BOBJ1	Span 2	10.35	Bent	6	9.238E-09	1.503E-09	5.154E-09	0.000083	0.003188
Seismic	BOBJ1	Span 3	23.45	Bent	1	2.940E-09	2.017E-09	5.435E-09	0.000727	0.007353
Seismic	BOBJ1	Span 3	23.45	Bent	2	2.428E-09	2.834E-09	5.836E-09	0.000676	0.007727
Seismic	BOBJ1	Span 3	23.45	Bent	3	2.225E-09	2.796E-09	6.031E-09	0.00063	0.008019
Seismic	BOBJ1	Span 3	23.45	Bent	4	2.225E-09	2.796E-09	6.031E-09	0.00063	0.008019
Seismic	BOBJ1	Span 3	23.45	Bent	5	2.428E-09	2.834E-09	5.836E-09	0.000676	0.007727
Seismic	BOBJ1	Span 3	23.45	Bent	6	2.940E-09	2.017E-09	5.435E-09	0.000727	0.007353
Seismic	BOBJ1	Span 4	36.55	Bent	1	3.163E-09	7.918E-10	6.256E-09	0.000421	0.008703
Seismic	BOBJ1	Span 4	36.55	Bent	2	1.995E-09	1.078E-09	6.760E-09	0.000407	0.009286
Seismic	BOBJ1	Span 4	36.55	Bent	3	2.065E-09	1.088E-09	6.937E-09	0.000375	0.009602
Seismic	BOBJ1	Span 4	36.55	Bent	4	2.065E-09	1.088E-09	6.937E-09	0.000375	0.009602
Seismic	BOBJ1	Span 4	36.55	Bent	5	1.995E-09	1.078E-09	6.760E-09	0.000407	0.009286
Seismic	BOBJ1	Span 4	36.55	Bent	6	3.163E-09	7.918E-10	6.256E-09	0.000421	0.008703
Seismic	BOBJ1	Span 5	49.65	Bent	1	2.689E-09	1.639E-09	6.671E-09	0.000391	0.009324
Seismic	BOBJ1	Span 5	49.65	Bent	2	2.898E-09	2.275E-09	7.225E-09	0.000382	0.010082
Seismic	BOBJ1	Span 5	49.65	Bent	3	2.718E-09	2.270E-09	7.488E-09	0.000352	0.010505

Table 4.5: Bridge Seismic Design - Support Bearing Demand – Deformations”Table Continued”.

Seismic	BOBJ1	Span 5	49.65	Bent	4	2.718E-09	2.270E-09	7.488E-09	0.000352	0.010505
Seismic	BOBJ1	Span 5	49.65	Bent	5	2.898E-09	2.275E-09	7.225E-09	0.000382	0.010082
Seismic	BOBJ1	Span 5	49.65	Bent	6	2.689E-09	1.639E-09	6.671E-09	0.000391	0.009324
Seismic	BOBJ1	Span 1	60.	Abutment	1	9.345E-09	1.186E-08	5.991E-09	0.008224	0.017141
Seismic	BOBJ1	Span 1	60.	Abutment	2	8.115E-10	1.592E-08	6.049E-09	0.009545	0.017031
Seismic	BOBJ1	Span 1	60.	Abutment	3	8.698E-10	1.572E-08	5.926E-09	0.009449	0.016656
Seismic	BOBJ1	Span 1	60.	Abutment	4	8.698E-10	1.572E-08	5.926E-09	0.009449	0.016656
Seismic	BOBJ1	Span 1	60.	Abutment	5	8.115E-10	1.592E-08	6.049E-09	0.009545	0.017031
Seismic	BOBJ1	Span 1	60.	Abutment	6	9.345E-09	1.186E-08	5.991E-09	0.008224	0.017141

5. CONCLUSION

5.1 CONCLUSION

In this research, the seismic analysis and seismic response of the bridge under traffic load were analysed in CSI Bridge 2.0, a specific bridge was considered for analysis. According to the AASTHO LRFD and HL-93 standards, a specific bridge measuring 60 metres in length and 15.5 metres in width was taken into consideration. The bridge was a four-lane, concrete tee-beam bridge with piles in the foundation that were each 10 metres tall. It was located in Russia.

3520.03 KN-M was found to be the maximum moment about the horizontal axis under load combo 1, and 4726.90 KN-M was found to be the maximum stress on the entire bridge section. The largest longitudinal displacement remained at $8.196\text{E-}04$ metres, and the maximum vertical displacement over the entire span of the bridge was calculated to be $1.197\text{E-}03$ metres.

Additionally, using the ASHTO LRFD 2012 code with built-in load cases, the pushover analysis was done to determine the seismic design response of the bridge. The longitudinal and transverse bends of bridge piers were examined. The highest value of displacement was 42.27mm in pushover curve 7 under load case PO TR4 for bent 4 in the transverse direction. Additionally, the displacement in Pushover curve 8's load case PO LG4 for bent 4's longitudinal direction was measured at 57.25mm.

Moreover, to determine how earthquake affect the normal frequencies and times of bridge observation, a modal analysis was performed. Six modes of deformed shapes' periods and frequencies were first studied. It was discovered that mode 5 with the deformed shape exhibits a useful response with the bridge's elastic property at $T= 0.12669$ seconds and $F= 7.89323$ Hz.

The following are some of the indicators that this study suggests.

- a. The pushover analysis shows that there is very little displacement with base shear, indicating that the bridge has excellent column bends in both the longitudinal and transverse directions.
- b. According to the modal analysis, the bridge's deformed shape exhibits a useful response to the elastic property of the bridge to its seismic response and bear the seismic wave.
- c. The dynamic analysis shows a fair response under various loading conditions and shows that, as compared to trucks and moving loads, different loads on a bridge exceed moment, stress, and displacement.
- d. The dimensions of the bridge, the outcomes of the modal, dynamic, pushover analysis, the values of the loads, the seismic response of the bridge under various modes, and the seismic response of the bridge under deformed shapes are all provided in this study, along with all other necessary requirements and specifications for the proposed bridge. Based on the particular code, the output from the CSI bridge 20 represents ideal values.

5.2 RECOMMENDATION

Based on the results of this study, it is recommended that analyses be conducted using various software programmes, including Safe Pro and Sap 2000, and different design codes, in order to understand the behaviour of structures. Additionally, to determine the stability of the structure under various circumstances as well as the behaviour of the structure under a wider range of distinct load cases and combination loads. Additionally, to conduct research on the structure's strength and cost - effectiveness under various loading scenarios and section configurations.

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